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Narrabeen-Collaroy Fishermans Beach

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Criteria for the Siting and Design of Foundations for Residential Development

Prepared for Warringah Shire Council

GEOMARINE Report No. 69021R02
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Preface

This report comprising recommendations for the siting of and foundation design specifications for residential beachfront development at Collaroy~Narrabeen~Fishermans Beach has been prepared jointly by **GEOMARINE P/L** and **Coffey Partners International Pty. Ltd.**

GEOMARINE is responsible for the broad concept and scope of the studies and has undertaken the site specific coastal process assessments, the hazard definition and the definition and mapping of the various stability zones referenced. It has recommended the broad scope of foundation types that are applicable for the various identified stability zones of the frontal dune. Specifically, therefore, **GEOMARINE** is responsible for the preparation of **Sections 1, 2, 3, 4.1 and 4.2.**

COFFEY has undertaken the field investigations and data acquisition, the laboratory testing programme and the analysis of foundation loadings leading to the preparation of the charts for foundation design. Specifically, therefore, **COFFEY** is responsible for **Sections 4.3, 4.4, 4.5, 5.1 5.2 and 5.3 and Appendices A,B and C.**

The study was designed and co-ordinated and the report was produced by **GEOMARINE.**

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Summary and Recommendations

This report presents a comprehensive description, assessment and definition of hazards relating to coastal beachfront development at Narrabeen~Collaroy~Fishermans Beach. The aim of the report is to provide comprehensive criteria for the appropriate siting and foundation design of new development.

The report presents a detailed assessment and quantification of coastal process factors at Narrabeen~Collaroy~Fishermans Beach. A stability assessment is presented that takes account of field data obtained from a drilling and *in-situ* testing programme undertaken along the frontal dune and laboratory data from grading and direct shear box tests.

The stability assessment has been used to define various zones requiring particular standards for building foundations and these have been mapped both within the report and on three sheets provided separately at 1:2,000 scale. The foundation design criteria have been developed for residential beach developments in each of the zones and the following recommendations are made:

- In the *Zone of Wave Impact* residential developments should not be constructed.
- In the *Zone of Slope Adjustment* pile foundations should be used and these should be designed to withstand the structural axial and lateral design loads in addition to the loads induced in the pile by soil "slumping" past the piles during severe beach erosion.

- In the *Zone of Reduced Foundation Capacity* either spread footings or piles can be used, but the foundations must generate adequate resistance within the underlying *Stable Foundation Zone* to resist the design axial and lateral structural loads.
- In the *Stable Foundation Zone* foundations can be designed by conventional means to resist the design axial and lateral structural loads.

Recommendations are made for:

- assessing the loads and moments induced in a pile by a slumping soil mass;
- the requirements for a geotechnical investigation; and
- assessment of the geotechnical parameters required for foundation design.

The results of drilling at eight sites in the Collaroy-Narrabeen Beach areas have been presented and design parameters for these specific areas have been developed.

Finally, design charts have been prepared to aid the calculations involved in the foundation design process.

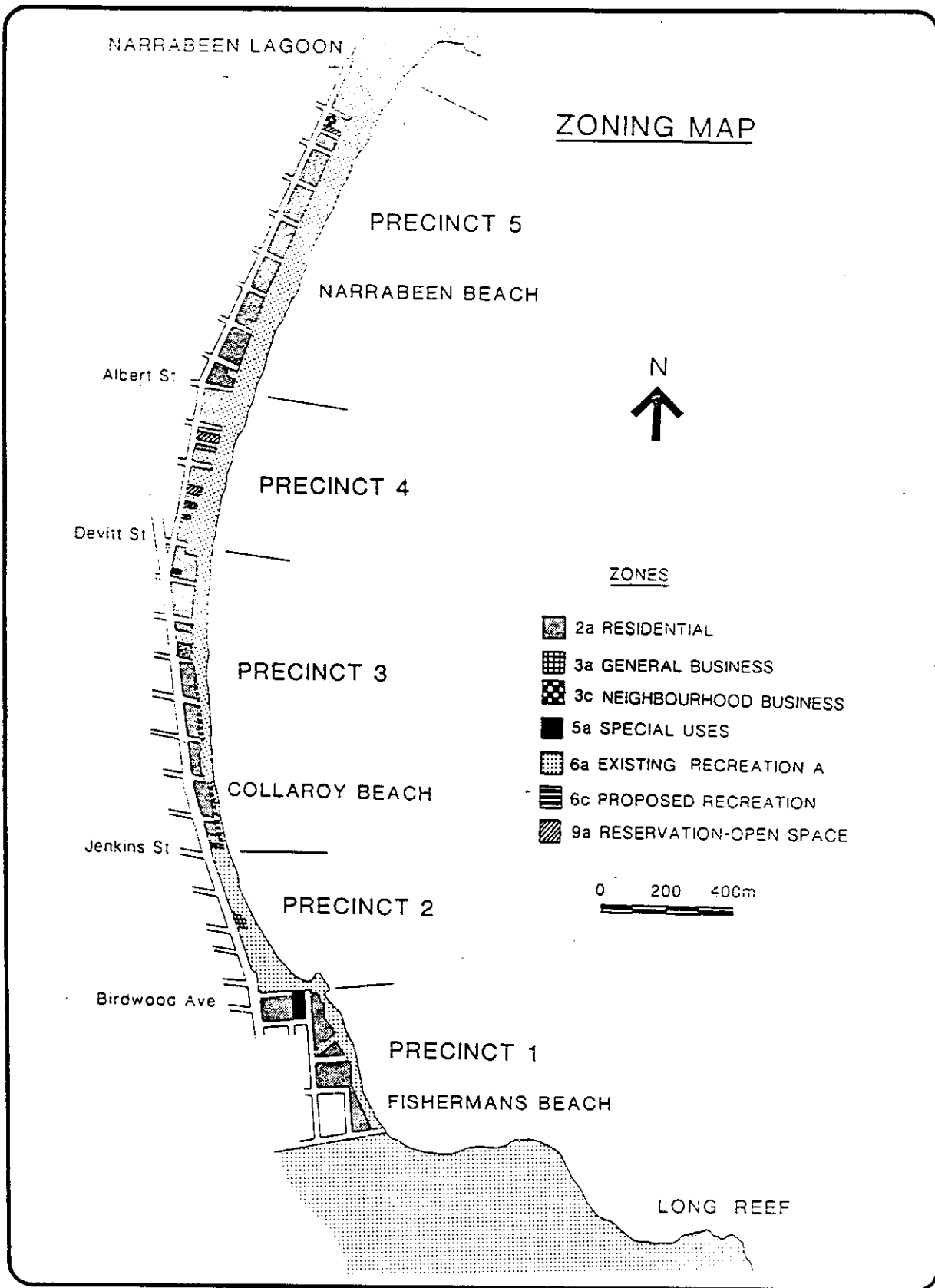
It is recommended that specialist geotechnical advice be sought in order to properly plan and execute the geotechnical investigations required for beach and coastal residential developments and carry out the geotechnical design of the foundations.

1. Introduction

Storm erosion presents a hazard to properties along Narrabeen-Collaroy Beach and Fishermans Beach (**Figure 1.1**). Studies to date undertaken by the Public Works Department (P.W.D., 1987) and Council (W.S.C., 1988; W.S.C., 1989) have shown that much of the beachfront development at Collaroy (south of Devitt Street) and a number of dwellings at Narrabeen are under threat from severe storm erosion. In the longer term the risk to properties at Collaroy-Narrabeen Beach is likely to increase should sand continue to be removed from the beach by natural processes and this threat may be exacerbated also should the prognosis of a sea level rise resulting from the *Greenhouse Effect* come to fruition.

Council is considering options for the long term management of the beachfront at Narrabeen-Collaroy Beach and Fishermans Beach. These options are aimed at maintaining and enhancing the recreational amenity of the beach and protecting property at risk. However, until such works are carried out Council cannot approve developments in hazardous areas without taking adequate precautions to ensure the safety of any new buildings. To this end Council has specified criteria comprising adequate setbacks and special foundation conditions for any new development in the hazardous areas that will ensure its safety until the longer term management options are implemented.

This report presents the details of the determination of building setbacks and foundation design criteria required for new residential development to ensure its safety and to allow Council to consider applications *in the interim* while exercising its *Duty of Care*.



Warringah Shire Council
 Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
Location Diagram

Reference: W.S.C., 1989

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**Figure
1.1**

COFFEY

2. Coastal Process Factors Relating to Foreshore Stability

2.1 Water Levels, Wave Runup and Seepage

2.1.1 Preamble

During storms the ocean water level and that at the shoreline is often higher than the normal tide level. While these higher levels are infrequent and last only for short periods they may exacerbate any storm damage on the foreshore. Elevated water levels allow larger waves to cross the offshore sand bars and reefs and break at higher levels on the beach.

The components of elevated storm water levels comprise astronomical, meteorological and global factors. All of the components do not act or occur necessarily independently of each other but their coincidence and degree of interdependence generally is not well understood.

BLAIN BREMNER & WILLIAMS P/L, Consulting Engineers, and **LAWSON AND TRELOAR P/L**, Coastal, Ocean and Port Consulting Engineers, in conjunction with **WEATHEREX Meteorological Services P/L** and on behalf of the Public Works Department, have carried out detailed studies of historical events which have caused elevated ocean levels on the N.S.W. coast (PWD Report Nos. 85041, 86005 and 86026).

2.1.2 Astronomical and Meteorological Components of Water Level

The storm water level depends primarily on:

- the prevailing astronomical tide;
- the intensity, scale, direction and speed of movement of the storm;
- the bathymetry of the coastal area including the presence or otherwise of offshore reefs and islands; and
- the shape of the coastline including the topography of the nearshore areas which may be inundated.

Tides

The astronomical tide, generated by the gravitational attraction of the moon, the sun and other planets, is the largest single component contributing to the total water level measured above a low water datum on the New South Wales coast.

On most days there are two high tides and two low tides (semi-diurnal tides), but not usually of the same levels. The tidal ranges vary significantly throughout each lunar month and from month to month. The Highest Astronomical Tide is 1.1m (approximately) above Australian Height Datum. The frequency of occurrence of the very high and low tides increases around Christmas time and in the mid-winter months.

Storm Surge

Storm surge is caused by the local rise in ocean level resulting from the drop in atmospheric pressure (the *inverse barometer* effect) and by piling up water against the shore as a result of the wind stress caused by the strong onshore winds generated by the low pressure system. For the low pressure system of the severe storms in May, 1974 the estimated central pressure was 986mb which would have resulted in a pressure surge of 0.28m above the predicted tide level.

The maximum wind speeds estimated for the 1974 storms of 90knots would have produced a wind set-up of approximately 0.19m along an open coastline (Lawson & Treloar, 1985).

Shoreline Wave Setup and Surf Beat

The breaking action of waves results in an increase in water levels, known as wave setup, in the surf zone. The physics of wave setup are described by BATTJES (1974) for the case of spilling breakers. Wave setup may be perceived as the conversion of part of the wave's kinetic energy into potential energy.

The amount of wave setup will depend on many factors including, among other things, the type, size and periods of the waves, the nearshore bathymetry and the slope of the beach.

In many cases waves tend to propagate in groups of large and then small waves. This wave grouping has the effect of inducing water level changes at the shore with periods in the order of several minutes and which are amplified in shallow water. These longer period water level fluctuations are often called surf beat and may have amplitudes of up to a few metres at the shoreline during severe storms.

Wave setup may be calculated using simplified methods found in the Shore Protection Manual (C.E.R.C., 1984) or by using computer methods where the offshore slopes are complex and natural wave spectra are being considered (after GODA, 1975). Some field measurements of wave setup at a number of locations on the New South Wales coast have been made by the Public Works Department (P.W.D., 1988; NIELSEN, 1988; DAVIS & NIELSEN, 1988).

On the New South Wales coast during severe storms wave setup can be some one to two metres and makes the largest contribution to elevated water levels above the tide. While the amount of wave setup varies according to the water depth a value of 1.4m has been adopted as a design wave set-up at the shoreline of Collaroy Beach (W.S.C., 1988; W.S.C., 1989).

Shelf Waves

Continental shelf waves cause medium-term variations in mean sea level of up to 0.2m over the time scales of days to weeks.

2.1.3 Global Processes

Very large scale processes acting over the globe causing significant water level variations include:

- tectonic changes;
- tsunamis;
- eustatic changes; and
- global-scale meteorological oscillations.

Tectonic Changes

Tectonic movements in the earth's crust over an extended area are often perceived as changes in ocean water levels. However, in New South Wales it is believed that there are no tectonic changes occurring which may affect sea levels along the coast.

Tsunamis

Tsunamis are caused by earthquakes on or near the sea bed and commonly, albeit misleadingly, are called *tidal waves*. Studies of the tide gauge records obtained at Fort Denison from 1867 have identified a number of water level anomalies resulting from tsunamis. The largest were 1.07m recorded in 1868 and 1877. More recently a tsunami resulting from a severe earthquake in Chile in 1960 caused the water level at Fort Denison to oscillate 0.84m over a 45 minute period. This rapid change induced very strong currents within the harbour, as well as in the other nearby ports and bays, causing considerable damage to boats and shoreline structures.

Eustatic Changes

Eustatic sea level changes are those resulting from changes in the volume of water frozen in the polar ice caps. While the geological history has many ice ages the ocean levels have remained relatively stable over the past 6,000 years. There are no confirmed indications of any significant change in the foreseeable future. However, there may be possible changes resulting from the *Greenhouse Effect*.

The term *Greenhouse Effect* is used to describe a postulated warming of the earth due to the accumulation in the atmosphere of certain gases, and in particular carbon dioxide resulting from the burning of fossil fuels.

The Board of Engineering of the Institution of Engineers, Australia issued a *Policy On The Greenhouse Effect* (1/5/15) in August, 1989. The policy statement, *inter alia*, expresses a recognition by the Institution of a need for engineering practice to take into account the potential impacts of the *Greenhouse Effect*. It states also that engineers have a professional responsibility to ensure that their activities reflect the best information available at the time. However, while the Institution has established a committee to prepare guidelines for the incorporation of the effects of climate change in coastal engineering design, to date no guidelines have been issued.

The current consensus of scientific opinion is that such changes could result in global warming of 1.5° to 4.5°C over the next 30 to 50 years. Such a warming could lead to a number of changes in climate, weather and sea levels. These in turn could cause significant changes to coastal alignments and erosion. Global warming may produce also a world-wide sea level rise caused by the thermal expansion of the ocean waters. The U.S. Na-

tional Research Council, 1987 presents a range of sea level rise scenarios of between 0.13m to 0.32m by the year 2036; that is 50 years from 1986 (Technical Memorandum 88/02 appended states GEOMARINE policy regarding eustatic sea level changes). It is predicted that the severity and frequency of storms will increase, rainfall intensity could increase and there could be a more severe wave climate. However, the likely degree of change in these processes has not been quantified. For hazard definition at Collaroy-Narrabeen Beach Council has adopted the following values:

TABLE 2.1 Values for Greenhouse sea level rise

Development	Planning Period	Sea Level Rise
Open space	25 years	0.05m
Residential	50 years	0.23m
Intensive development	100 years	1.17m

El Nino Southern Oscillation

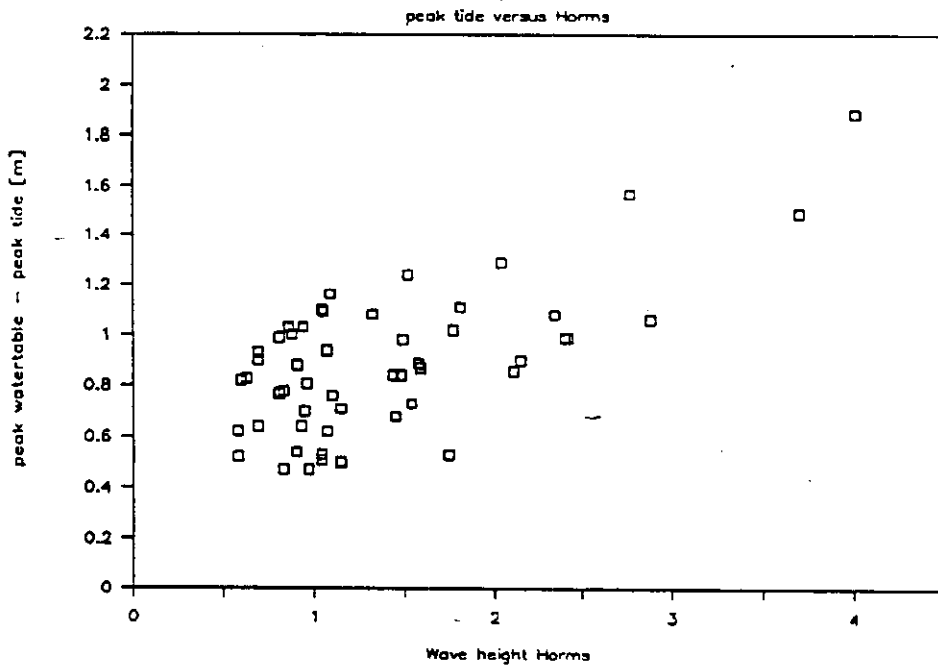
Global meteorological and oceanographic changes such as the El Nino Southern Oscillation in the eastern southern Pacific Ocean cause medium-term variations in mean sea level of up to 0.1m.

2.1.4 Wave Runup

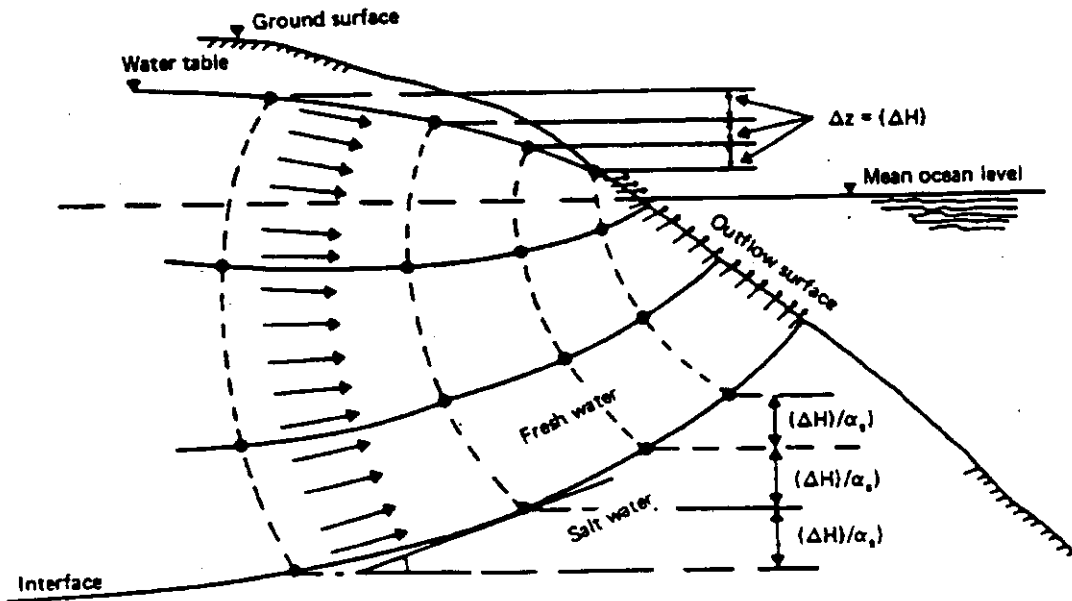
The energy of a wave is dissipated finally as the water runs up the beach or shoreline. Wave run-up is the vertical distance the wave will reach above the level of the tide, storm surge and wave setup and can be several metres.

Wave run-up at any particular site is very much a function of the beach profile, the surface roughness and other shoreline features on which the breaking waves impinge. Physical model results for run-up levels are available in the Shore Protection Manual for simple profiles and wave conditions. At Collaroy Beach from *The Breakers* to *The Marquesas* wave run-up from R.L. 7m to 8m (respectively) has been adopted for hazard definition.

Peak Watertable super elevation above



Super-elevation of peak dune water table due to various incident wave conditions (P.W.D., 1988).



Hydraulic conditions at the ocean interface resulting from freshwater surcharge of the dune (Kashef, 1987).

Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
Groundwater and Flow Conditions in a Sand Dune

Figure 2.1

2.1.5 Seepage

The water table inside the beach will follow the movements of the swash zone up and down in response to changing tides and wave conditions. When storms raise the average water levels on the beach above those of the normal tide the levels in the dune may rise correspondingly. The super-elevation of peak water tables in the dune above the peak tide level has been measured by the Public Works Department for some wave conditions as shown here in **Figure 2.1**. Should the beach water levels fall rapidly, say with a drop in wave height on the beach with a falling tide, the water table in the dune may remain elevated for some time as the water slowly seeps out (**PUBLIC WORKS DEPARTMENT, 1988**). While under normal tide conditions the water table in the beach may be elevated some 0.5m above the average water level, during storms and on the falling stages of the tide the differences may be several metres. As shown in **Figure 2.1** also, heavy groundwater inundation of fresh water may become perched on top of the saline wedge with seepage occurring over the beach section to depth.

Seepage forced by water table gradients may reduce the effective frictional shear strength of soils, thereby lowering considerably the stability of the dune. The generalised case of parallel seepage through a uniform sand slope has been solved and indicates a reduction in the stable maximum slope to about half that for no seepage (**LAMBE & WHITMAN, 1969**). In the case of a rapid drawdown in the water level on the beach the maximum seepage pressures may be expected at the toe of the dune escarpment, reducing significantly the stability of the overall dune face. Heavy rainfall often associated with severe storms may cause ponding behind a frontal dune, thereby increasing the seepage pressures at the toe of the dune escarpment. Elevated water tables will reduce also the foundation loading capacity.

2.2 Beach and Dune Erosion

The amount of sand eroded from the beach berm and foredunes will have a significant bearing on the stability of the foreshore in its eroded state. The frontal dune (or foredune) is part of the beach that can become mobilised during storms. It is likely that a severe storm will erode the beach back into the toe of the frontal dune. While the face of an accreted frontal dune typically may have a relatively low grade down to the beach as a result of the build-up of the incipient foredunes at its toe, during times of severe storm erosion the dune face will be cut and will stand

up at very steep angles. At this time the dune face is unstable and likely to slump.

The few measurements of storm sand erosion (*Storm Erosion Demand* in cubic metres of sand above A.H.D. for each metre of beach length; i.e. m^3/m) from open coast beaches on the Australian eastern seaboard include:

- $200m^3/m$ at Narrabeen Beach (P.W.D., 1987) for the storms in 1974;
- $240m^3/m$ at Wamberal Beach (P.W.D., 1985b) for the storms in 1974;
- $200m^3/m$ at Avoca Beach (GOSFORD CITY COUNCIL, 1989a) for the storms in 1974;
- $275m^3/m$ at Copacabana Beach (GOSFORD CITY COUNCIL, 1989b) for the storms in 1974;
- $200m^3/m$ for the storms in 1974 and $250m^3/m$ for the storms in 1986 at Forresters Beach (GOSFORD CITY COUNCIL, 1990);
- $190m^3/m$ at Terrigal Beach for the 1978 storms (P.W.D., 1984 & 1985b);
- a total of $320m^3/m$ from Iluka Beach during the storms of May and September, 1977 (P.W.D., 1985a);
- $170m^3/m$ to $430m^3/m$ from the Gold Coast beaches and dunes during the 1967 storms (McGRATH, 1968); and
- $260m^3/m$ to $320m^3/m$ from the beach and dune at Byron Bay over the period 23rd June, 1987 to 8th February, 1990 and encompassing the two severe storms of December, 1988 and April, 1989 (P.W.D., 1990).

For the stability assessment of the frontal dune at Collaroy/Narrabeen Beach an allowance of $200m^3/m$ (at *The Breakers*) to $250m^3/m$ (at *The Marquesas*) of sand erosion above A.H.D. has been made for storm erosion (to be applied to the average of available beach profile data).

The P.W.D. study of Fishermans Beach presented no data relating to *Storm Erosion Demand*. While photogrammetric data similar to that for Collaroy-Narrabeen were available for Fishermans Beach they showed no erosion from the 1974 storms; in fact they showed accretion. In the absence of any measured erosion on the beach the **Hazard Definition** study advised a value of $100m^3/m$ for *Storm Erosion Demand*. However, since that study additional data have become available for

storm erosion on such protected beaches which allows for a revision of the earlier estimates.

GEOMARINE has completed recently studies of storm erosion on some somewhat protected beaches such as Pearl Beach and Ocean/Umina Beach for the Gosford City Council (**GOSFORD CITY COUNCIL, 1990b & c**, in prep.). For these studies wave data were available from direct field measurements as presented in the **GEOMARINE (1988b)** report on the coastal processes of Broken Bay. At Pearl Beach, where wave height coefficients are some 70% of those for open ocean beaches such as Collaroy-Narrabeen, a storm erosion demand was measured at $125\text{m}^3/\text{m}$ at the most exposed (northern) section of the beach (see **GOSFORD CITY COUNCIL, 1990b**). For Ocean/Umina Beach a reduction to approximately 60% in the wave coefficients was correlated to actual storm erosion of $100\text{m}^3/\text{m}$ (see **GOSFORD CITY COUNCIL, 1990c**).

These data verify earlier analytical assessments of the relationship between storm erosion and wave height proposed in the Public Works Department's assessment of erosion at Boomerang Beach (**P.W.D., 1985a**); that is, subaerial beach storm erosion being a direct function of wave height squared (for a specific storm duration).

On the basis of wave studies undertaken at Collaroy-Narrabeen-Fishermans Beach (**GEOMARINE, 1988a**), the unrefracted deepwater significant design wave height parameter for Fishermans Beach is some 40% to 45% (SE to NE respectively) of that for Narrabeen Beach where the *Design Storm Erosion Demand* is some $250\text{m}^3/\text{m}$ (for SE storms). On the wave energy basis as defined above, therefore, a *Storm Erosion Demand* for Fishermans Beach can be evaluated analytically and is $40\text{m}^3/\text{m}$ to $50\text{m}^3/\text{m}$, which is higher than the maximum variation measured on the beach over the 45 year record. We recommend that Council adopt now $50\text{m}^3/\text{m}$ for the *Design Storm Erosion Demand* in lieu of the $100\text{m}^3/\text{m}$ as provided originally in the design parameters of the **Hazard Definition** study, and we propose to adopt this value now for the current exercise in defining the hazard zones for Fishermans Beach. However, we see no reason to vary the parameters adopted for Collaroy-Narrabeen Beach at this stage.

During severe storms the beach berm will scour below the low levels that are observed on the low tides immediately following storms. While sand is mobilised and taken into suspension under wave action at the toe of the dune escarpment, it is not

necessarily removed following the passage of a wave and may leave the impression that the level of erosion is above that which actually occurs. Following storms, the beach berm begins to recover very quickly and even within a few hours of lower swell conditions the amount of sand deposited at the top of the swash can be considerable. The depth of mobilisation will be below any post-storm measurements or observations.

Berm levels on natural beaches following storms have been measured as low as 0.5m A.H.D. However, drilling on Narrabeen Beach has indicated erosional disconformities in the beach sands in front of the natural dune at a level of -1m A.H.D. (GEOL. SURVEY N.S.W., 1982). Drilling at the back of the beach berm at Wooli, northern N.S.W. shows disconformities consistently at a level of approximately -1.0m A.H.D. (GEOL. SURVEY N.S.W., 1985). Two boreholes taken at the back of the beach berm on Dee Why Beach indicated beach sand overlying organic fill and sandy clay at approximately -0.8m and -1.5m A.H.D. (PUBLIC WORKS DEPARTMENT, 1977). These data indicate consistently a scour level of -1.0m A.H.D. (approximately) on open ocean beaches. For the determination of hazard zones on Collaroy-Narrabeen Beach a scour level of -1.0m (A.H.D.) has been adopted with a level of 0m (A.H.D.) for Fishermans Beach.

2.3 Coastal Process Design Parameters

The coastal process design parameters adopted for defining the hazard at Collaroy-Narrabeen Beach and Fishermans Beach as discussed above are summarised in Table 2.2.

Hazard Definition Parameters

Precinct	P1	P2	P3	P4	P5
Parameter					
Storm Erosion (m ³ /m/annum)	50	200	250	250	250
Wave setup (m)	0.8	1.3	1.4	1.5	1.6
Inundation Level (m on A.H.D.)	6	7	8	9	9
Beach Scour Level (m on A.H.D.)	0.0	-1.0	-1.0	-1.0	-1.0

3. Stability Assessment

3.1 Introduction

Stability computations can serve as a guide to determining safe setback distances on frontal dunes that are prone to wave attack and slumping during storms. A generalised stability assessment has been carried out for the foreshore areas composed of unconsolidated dune sand based on field data comprising drilling and in-situ field testing (**Appendix B**) and soil properties determined from laboratory testing (**Appendix C**).

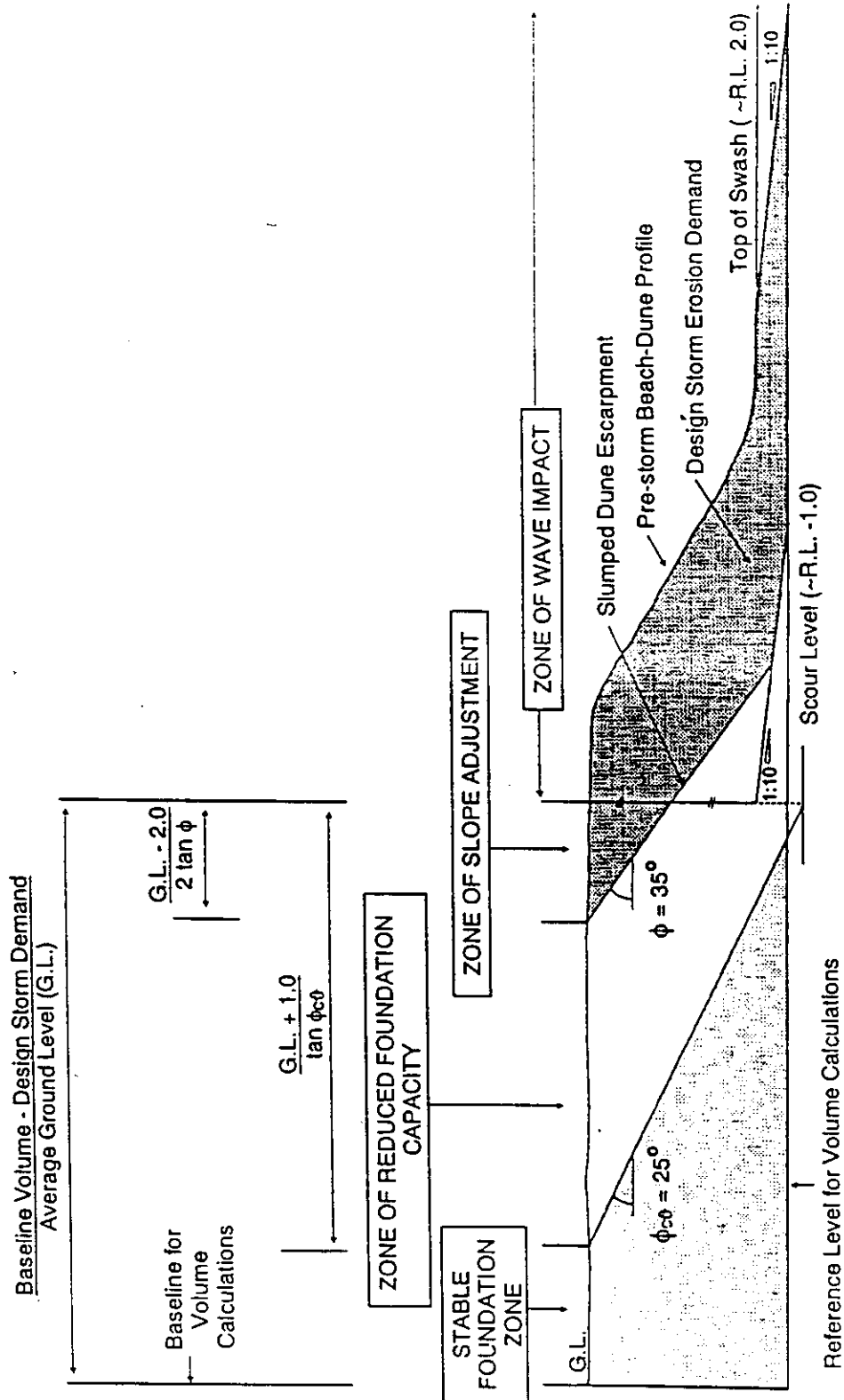
3.2 Factors Of Safety

The greatest uncertainties in stability problems arise in the selection of the pore water pressure and strength parameters. The error associated with the method of analysis, in the order of 10% in computed factors of safety for the better available techniques, is small compared to that arising from the selection of strength parameters. This is the reason for applying factors of safety to stability analyses (**LAMBE & WHITMAN, 1969**).

Where there is sound and detailed knowledge of the soil stratigraphy, as determined from drilling, and a detailed assessment of the strength of the various soil strata, as determined from in-situ field testing and laboratory testing, a factor of safety of 1.5 with respect to shearing strength is commonly adopted and has been adopted for this study.

3.3 Extent of Hazard

The application of the stability assessment is presented schematically in **Figures 3.1**. A number of zones are delineated; a *Zone of Wave Impact*, a *Zone of Slope Adjustment*, a *Zone of Reduced Foundation Capacity* and a *Stable Foundation Zone*.



Safe angle of repose of dune sand $\phi_c0 = \tan^{-1}((\tan \phi)/1.5)$

Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development

Schematic Representation of Stability Zones

Figure
3.1

The *Zone of Wave Impact* delineates an area where any structure or its foundations would suffer wave attack during a severe storm. It is that part of the beach that is seaward of the dune erosion escarpment.

A *Zone Of Slope Adjustment* was delineated to encompass that portion of the seaward face of the dune that would slump to the natural angle of repose of the dune sand following removal by wave erosion of the *Design Storm Erosion Demand*. That presents the steepest stable dune profile under the conditions specified.

A *Zone Of Reduced Foundation Capacity* for building foundations was delineated to take account of the reduced bearing capacity of the sand adjacent to the dune erosion escarpment. It was considered that structural loads should be transmitted only to soil foundations outside the zone within which the Factor of Safety was less than 1.5 during extreme scour conditions at the face of the dune. This allows for the design assumption that the soil may develop its full bearing capacity.

Structures not piled and located within the *Zone of Slope Adjustment* and seaward may be subject to foundation failure (soil slip and subsequent undermining) associated with a severe storm erosion event. Foundations for structures within the *Zone of Slope Adjustment* should comprise piles embedded to a sufficient depth within the *Stable Foundation Zone* to develop within that zone adequate capacity to support the vertical loads applied by the structure and differential horizontal soil pressures on the piles from the soil above the wedge failure plane.

For the same conditions, landward of the *Zone of Slope Adjustment* structures not adequately piled would be founded in a zone that could be considered to have an inadequate factor of safety (*Zone of Reduced Foundation Capacity*). Foundations for structures within this zone should comprise piers embedded to a sufficient depth within the *Stable Foundation Zone* to develop within that zone the capacity to support the vertical loads applied.

3.4 Methods of the Calculations

The basis for the calculations comprised the hazard definition design parameters defined in Chapter 2 and the photogrammetric survey data obtained for Council by the Public Works Department. Sand volumes along the beach were calculated

initially by the Department for the following dates of survey photography; 1941, 1951, 1956, 1961, 1970, 1972, 1974, 1985, and 1986. These are presented as cross-sectional areas of the dune and beach at shore-normal (approximately) profile lines spaced at 20m centres along the beach; the area being calculated as that area of cross-section above a datum of 0m (A.H.D.) and seaward of an arbitrary baseline and to the 2m beach contour (see **Figures 3.2, 3.3 and 3.4**).

The sand volumes calculated from each year of photography were averaged both spatially and temporally. The calculations were undertaken at the following locations (refer **Figures 3.2 to 3.4**):

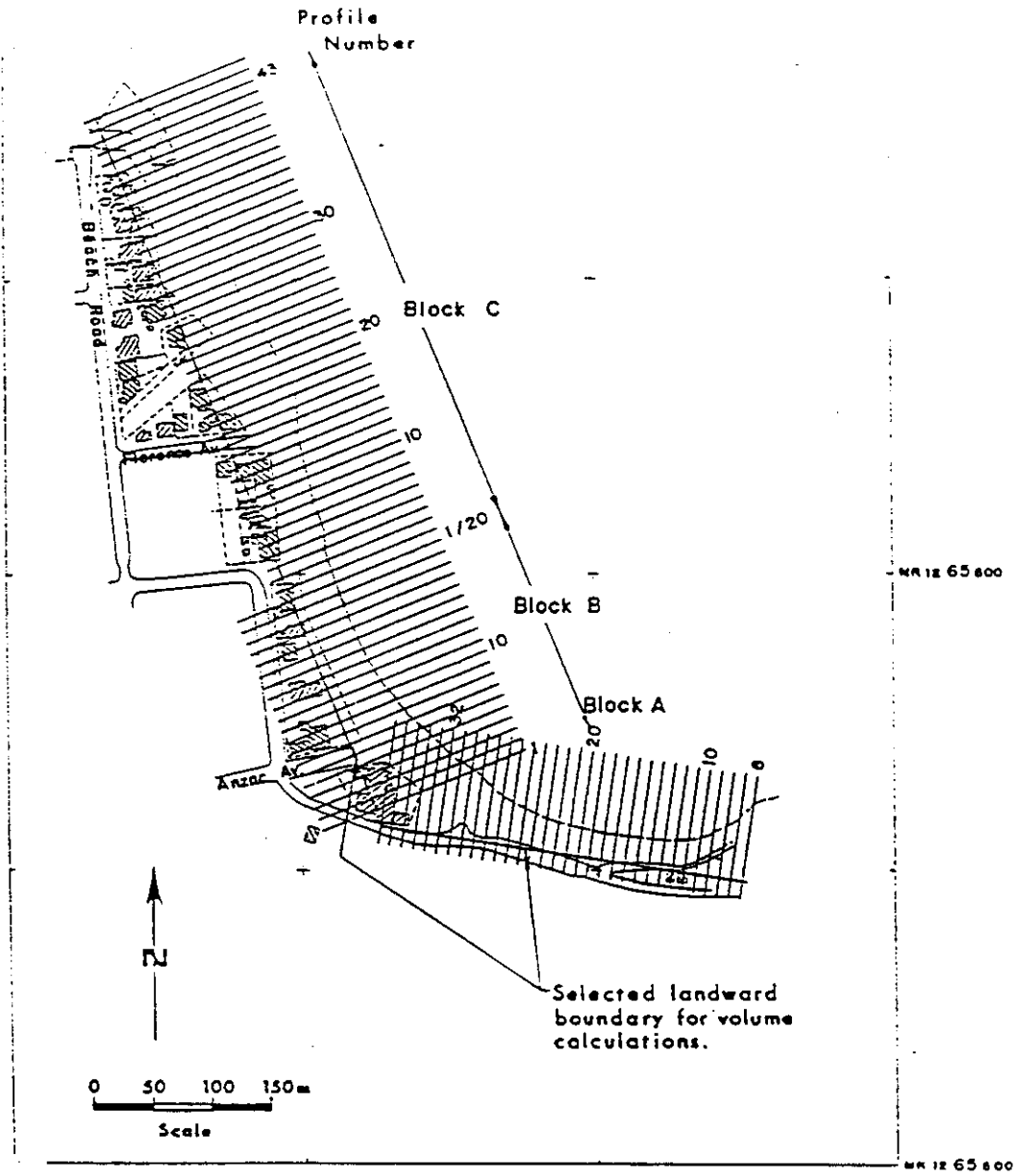
Block B	Profiles	10-13;
Block C	Profiles	1-4; 10-13; 23-26;
Block D	profiles	9-12; 30-31;
Block E	Profiles	1-4; 19-22 22-25 32-35
Block F	Profiles	13-16; 25-28;
Block G	Profiles	6-9; 24-27; 50-53.

At each location the profiles were averaged. For Collaroy-Narrabeen Beach the average volume thus calculated for each year then was discounted for the assessed long term erosion rate of $1.5\text{m}^3/\text{m}/\text{annum}$, the oldest profiles therefore being discounted the most. The average sand storage volume for the period of record was determined by summing the quotients of each of the discounted sand volumes with the average time period between each of the preceding and following surveys (for example, given that photography was available in 1941, 1951, 1956, . . . the time period quotient for the average sand volume for 1951 is $5 + 2.5 = 7.5$ years) and dividing the sum of these quotients by the period of survey record (1941 to 1991 = 50 years). For Fishermans Beach, while the same general procedure was applied, there was no discounting of sand volumes because the Public Works Department studies indicated that there was no long term sand loss from Fishermans Beach.

MR 12 65 000

MR 12 65 000

MR 12 66 000



NOTES

1. Base drawing is a photogrammetric plot from 7.4.1985 aerial photography. Structures shown are those visible in 1985.
2. Base drawing is derived from vertical aerial photography and therefore represents roof outlines of structures which generally do not coincide with the walls of the structures or the "building line".

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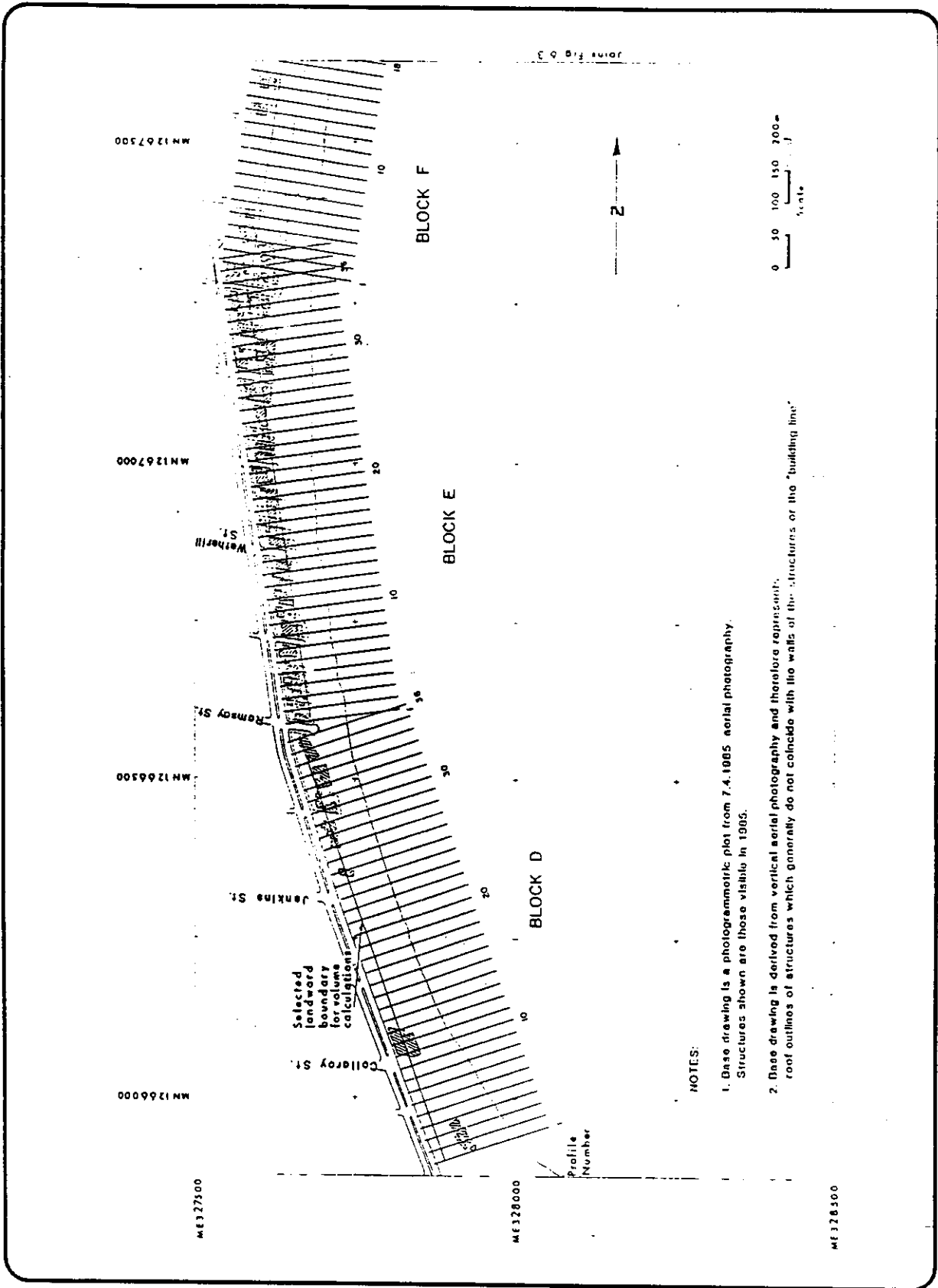
Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Profile Locations Fishermans Beach

Reference: P.W.D., (1987).

COFFEY

Figure 3.2

GEOMARINE



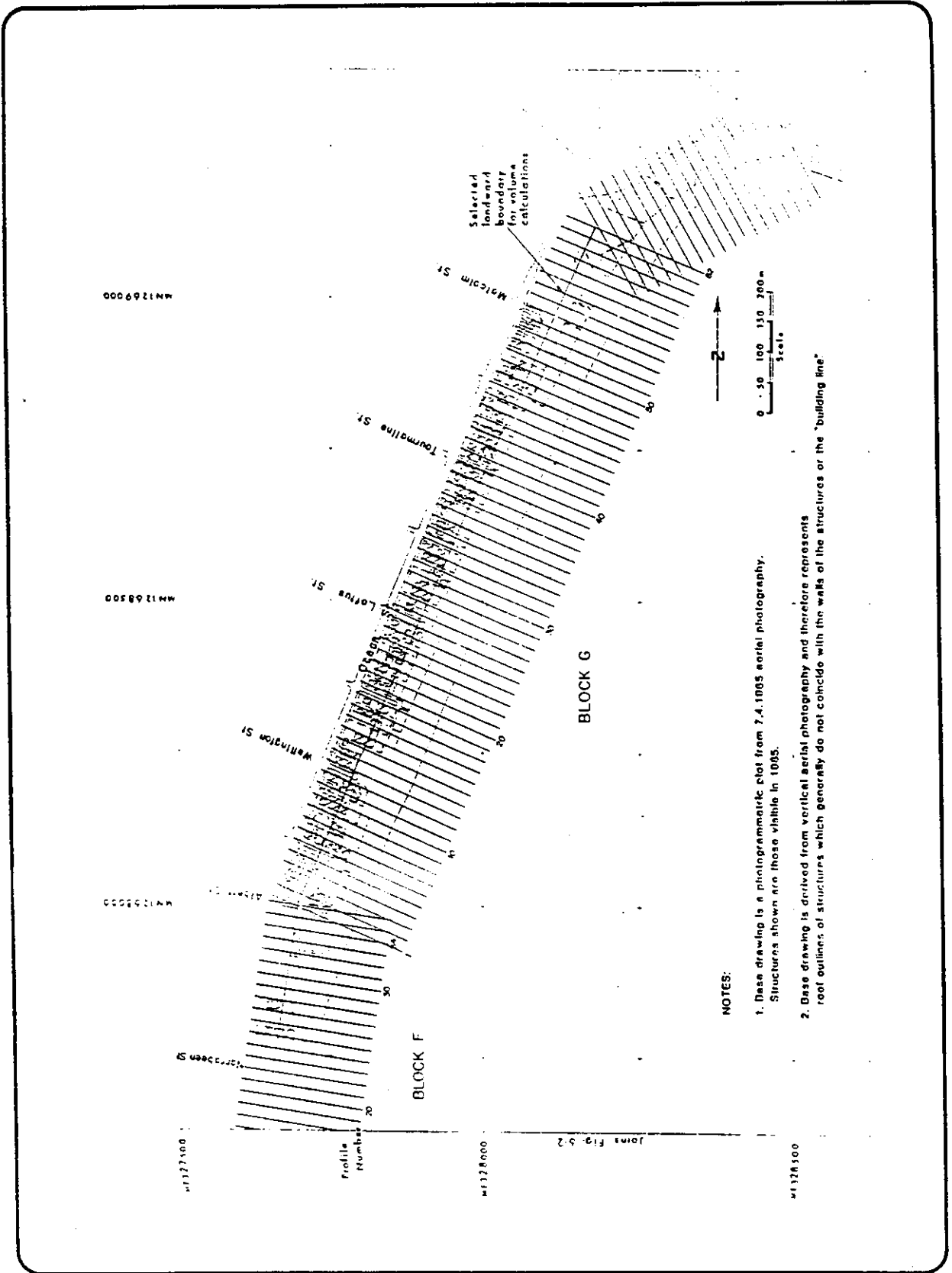
Warringah Shire Council
 Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Profile Locations Collaroy Beach

Reference: P.W.D., (1987).

GEOMARINE

Figure
3-3

COFFEY



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Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Profile Locations Narrabeen Beach

Reference: P.W.D., (1987).

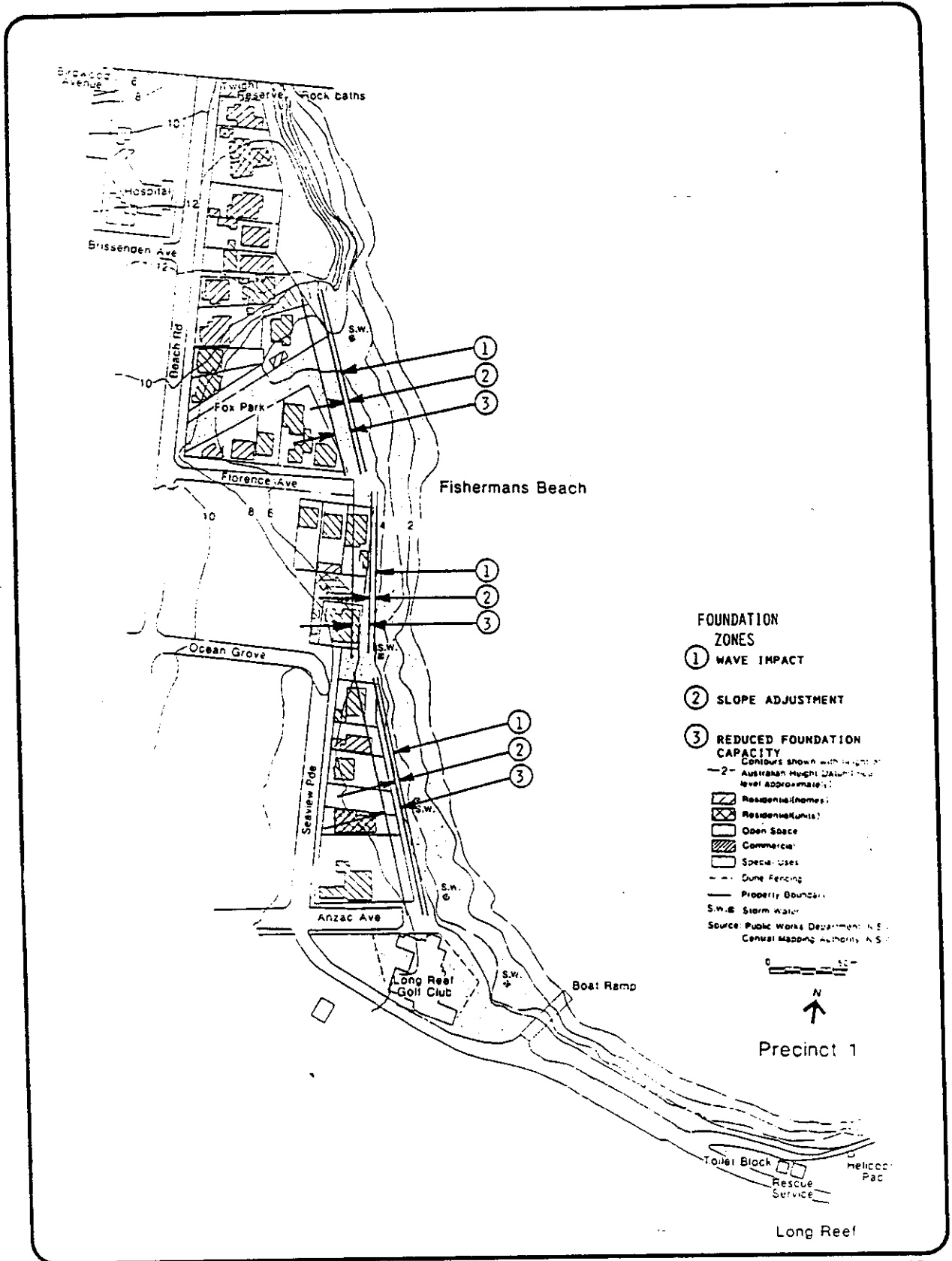
COFFEY

Figure
 3.4

GEOMARINE

To determine the extent of *Immediate Impact*, the storm erosion demand is subtracted from the available sand storage (**Figure 3.1**). The storm erosion profile is idealised as comprising a steep dune escarpment at a slope of the natural angle of repose of dune sand to the top of the wash zone at low tide, taken to be 2m (approximately on A.H.D.), then a steep nearshore beach face of slope 1:10 down to 0m (A.H.D.), the datum for the reference volume calculations. The distance from the top of this storm profile to the baseline is determined by dividing the difference between the baseline volume and the design storm erosion demand by the average ground level existing at the top of the dune seaward of the baseline, and subtracting from that an allowance for the slumping of the dune escarpment to its natural angle of repose as determined from the height of the dune face (Ground level (A.H.D.) - 2m(A.H.D.)) divided by twice the value of the tangent of the friction angle (ϕ) of the sand (**Figure 3.1**). A flatter dune face slope extending landward from the limit of beach scour and incorporating a *Factor of Safety* of 1.5 ($\tan \phi_{cv} = \tan \phi / 1.5$) defines the limit of the *Zone of Reduced Foundation Capacity* beyond which surface footings can be used safely.

The delineations of the various foundation zones determined in this way are presented in **Figures 3.5 to 3.11**. We note here that these Figures present only the hazard definition existing at present and do not take account of the longer term scenario of future progressive erosion of the beach sands resulting from the identified long term beach movements and the likelihood of a *Greenhouse* sea level rise.

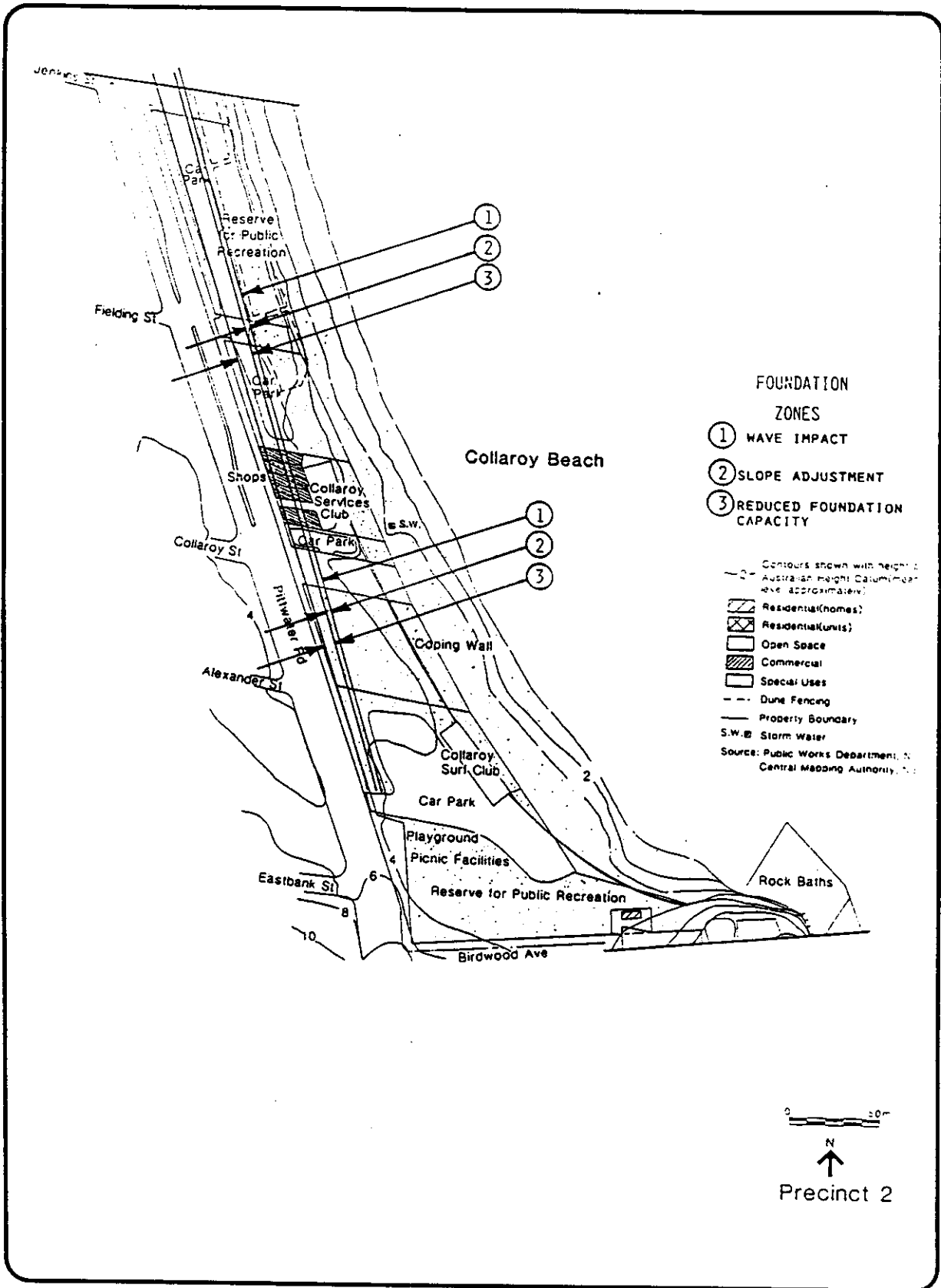


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 Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Stability Zones
 Precinct 1

Figure 3.5

COFFEY

GEOMARINE



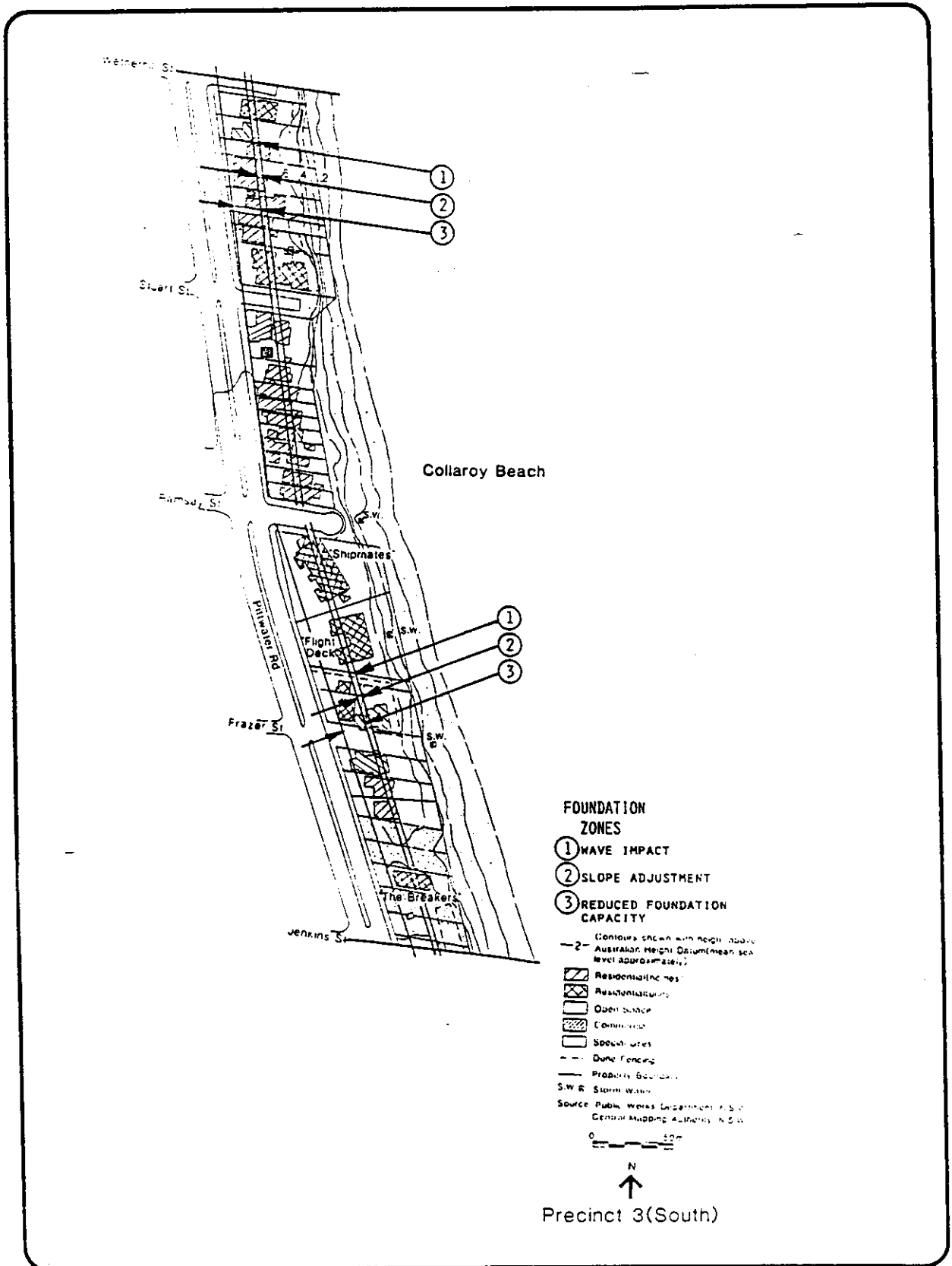
Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
Stability Zones
Precinct 2

GEOMARINE

Figure
3.6

COFFEY

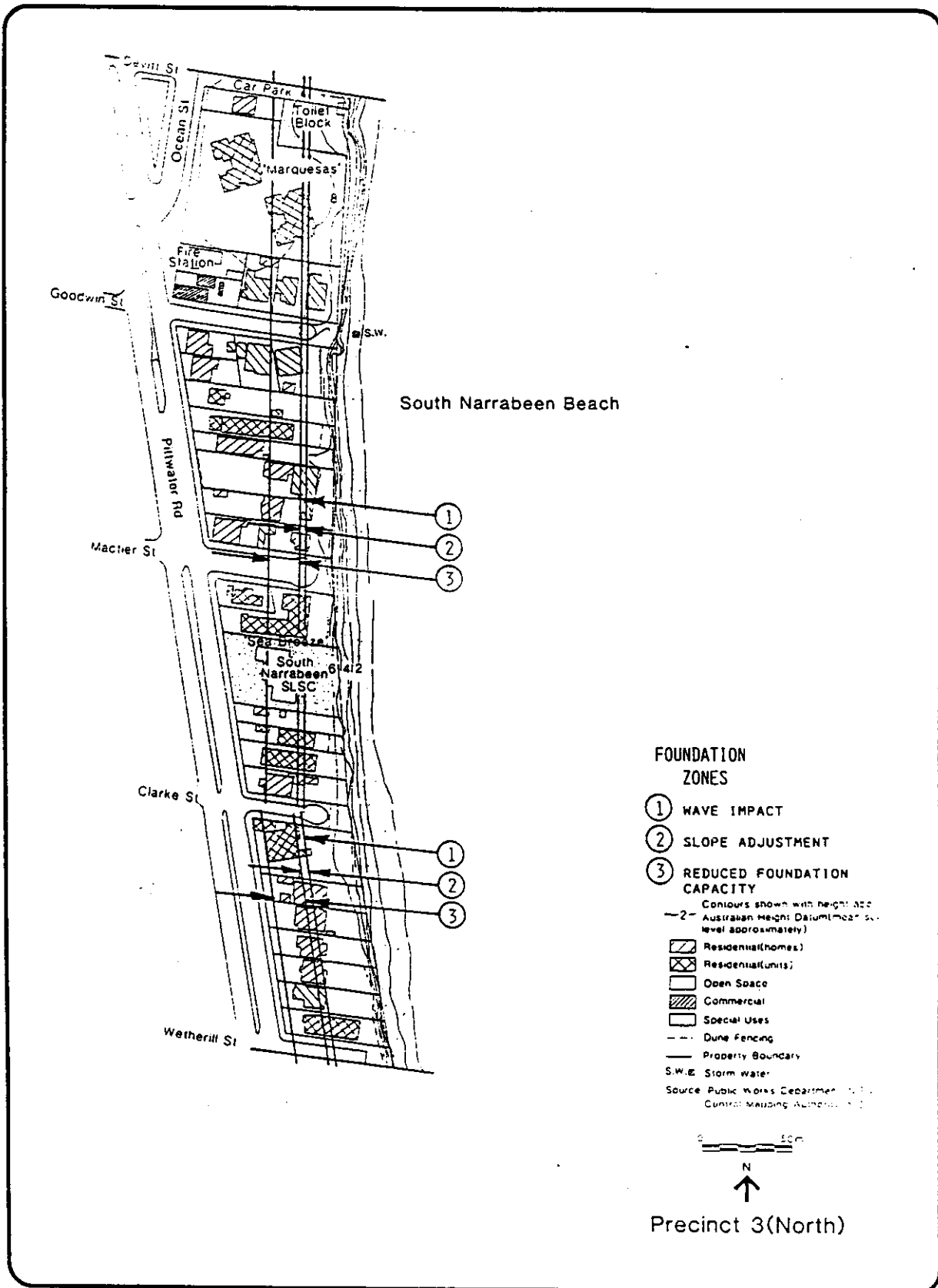


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 Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Stability Zones
 Precinct 3 (South)

Figure 3.7

COFFEY

GEOMARINE



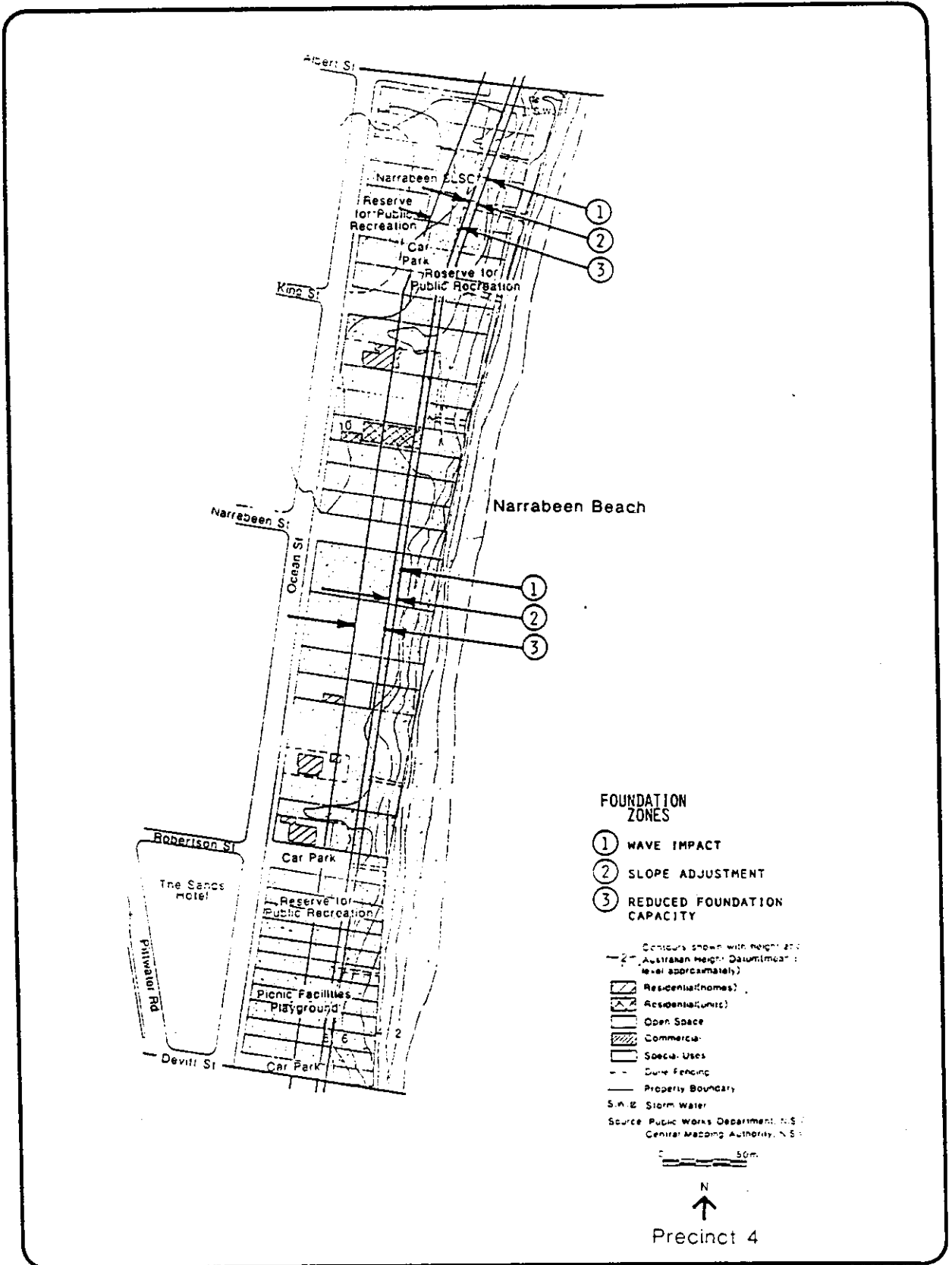
Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
Stability Zones
Precinct 3 (North)

GEOMARINE

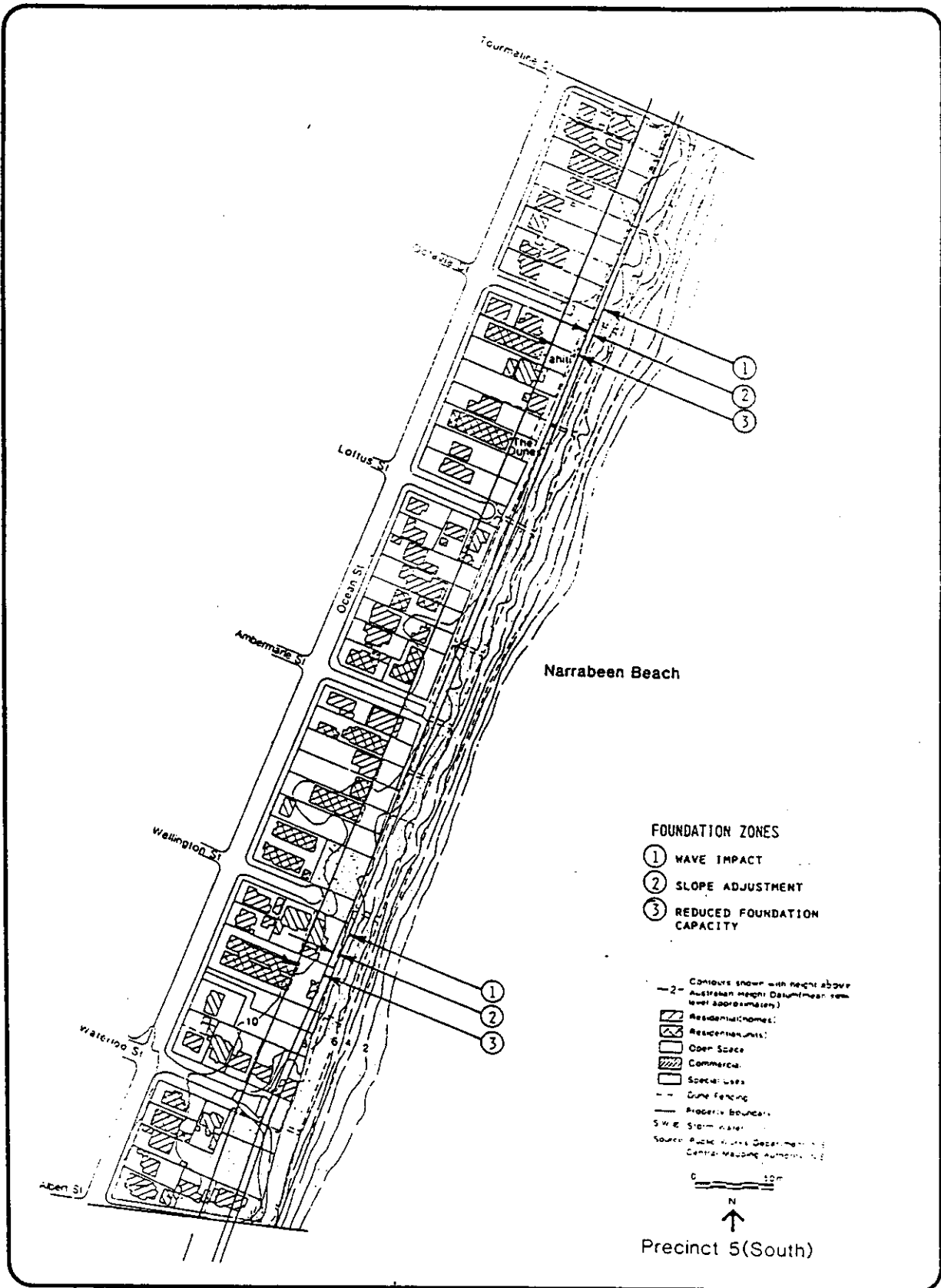
Figure
3.8

COFFEY



Warringah Shire Council
 Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Stability Zones
 Precinct 4

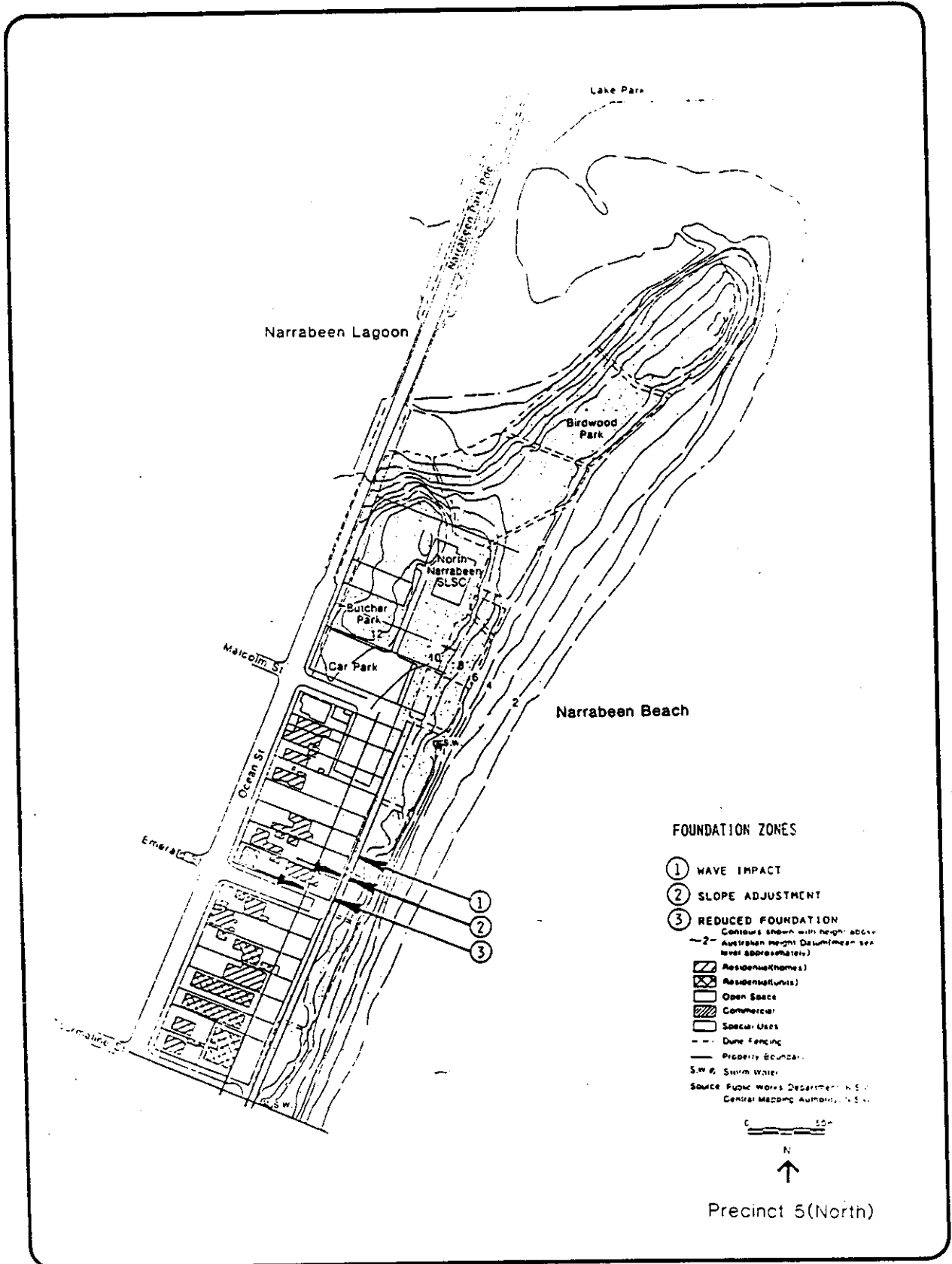
Figure
3.9



Warringah Shire Council

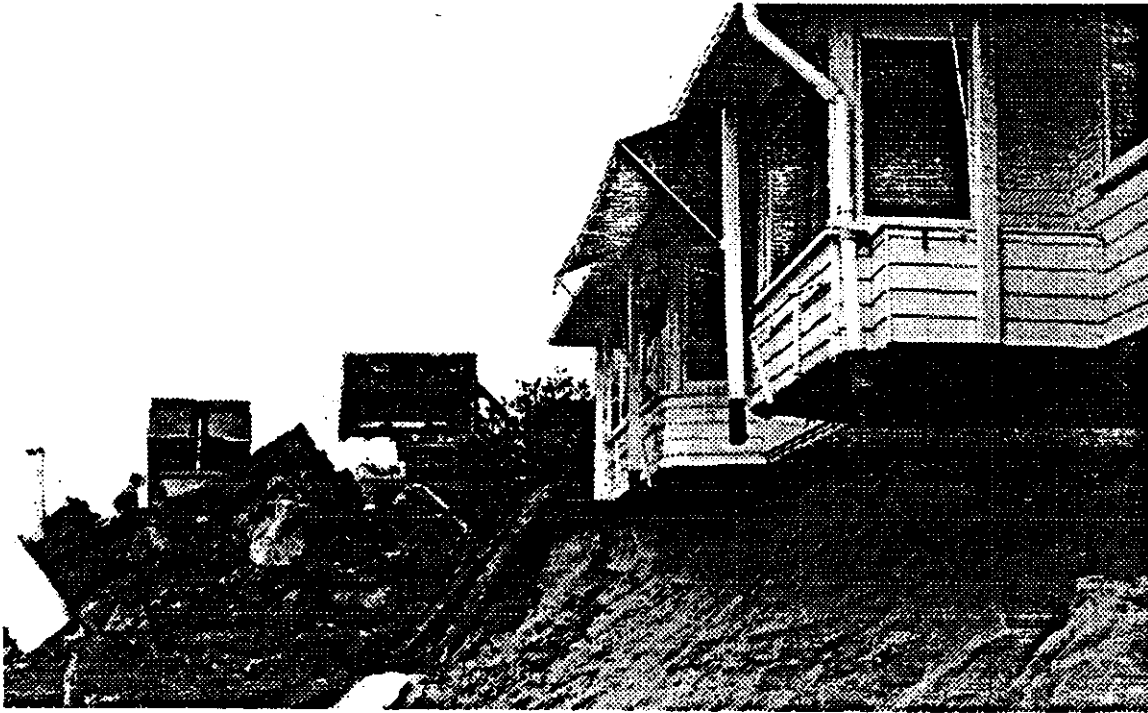
Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
Stability Zones
Precinct 5 (South)

Figure
3.10



Warringah Shire Council
 Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Stability Zones
 Precinct 5 (North)

**Figure
 3.11**



Warringah Shire Council

**Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development
1967 Storm Damage
Collaroy**

**Plate
4.1**

4. Considerations for Foundations of Single Residential Dwellings

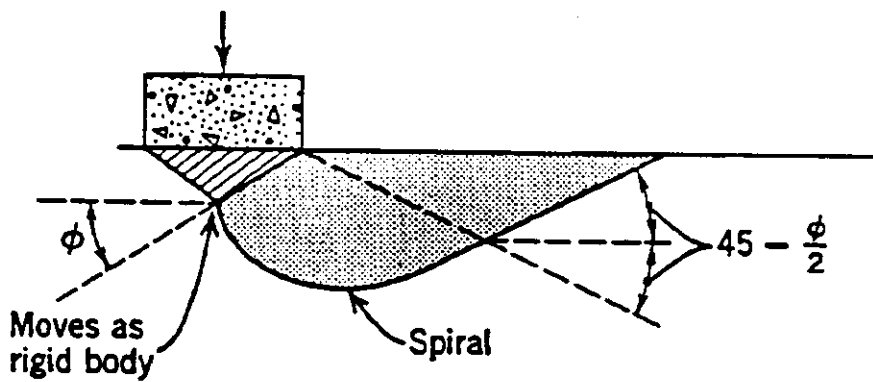
4.1 Introduction

Many domestic beachfront structures (homes) that have been destroyed during storms in New South Wales were lost as a result of failure of the soil mass rather than from direct wave impact loadings on the foundations. The inadequacy of standard foundations for houses located at the forefront of dunes is exemplified in **Plate 4.1** which shows the shoring up required to prevent the collapse of a dwelling at Collaroy during the storms of 1967.

There are no Standards Association of Australia codes of practice for the design and/or construction of foundations for domestic buildings in the active zone of the beach. The Institution of Engineers, Australia provides no guidelines nor is there any local government building code for such foundations in New South Wales. While there are engineering structures such as loading wharves and jetties designed for such locations, these are of a commercial nature or associated with defence and can bear a risk greater than that which is appropriate for domestic structures.

In determining appropriate design conditions for domestic foundations we consider it unacceptable that piling be subject to wave loading in the surf zone.

The factors that come under consideration in the design of structures on dunes include the extent of dune scour, the zone of slope adjustment and the structure foundation/soil interactions.



(after Terzaghi and Peck, 1967)

Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development

**Failure Zones for
 Shallow Footings**

**Figure
 4.1**

4.2 Types of Foundations

4.2.1 Shallow Foundations

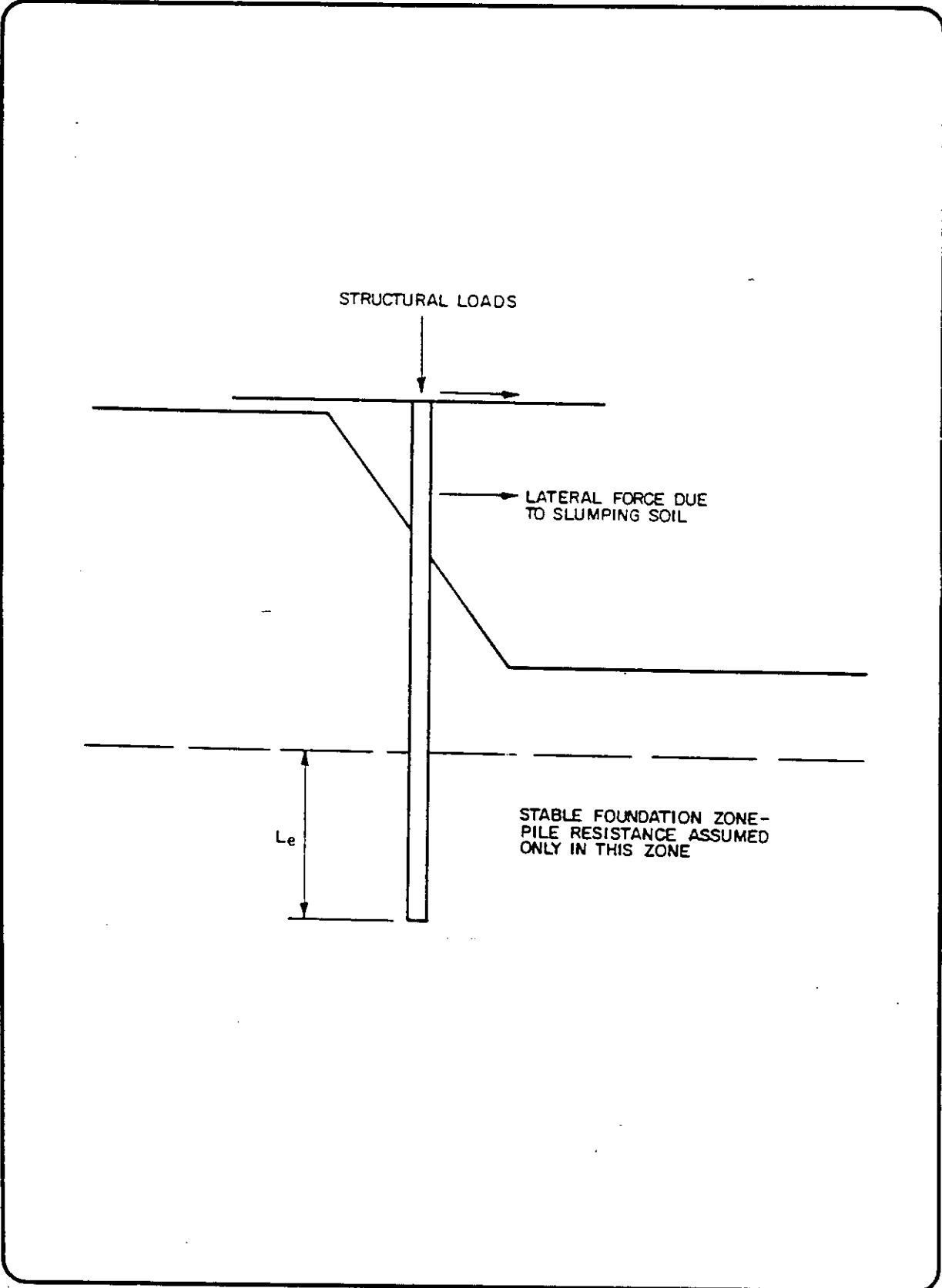
Shallow foundations include piers, strip footings and slabs. In common building standards the bearing capacity of such footings has been determined generally for the case of a horizontal bed. The failure zone of a shallow footing extends laterally (**Figure 4.2**) and, for shallow foundations located at or near the ground surface, should such a zone be located within the zone of potential slip failure then the bearing capacity of the footing would be reduced according to the extent of the zone of interference. To allow the soil to develop its ultimate bearing capacity such a footing would need to be set back such that the failure lines were beyond the failure zone of the stability analysis, that is, outside the design slip surface having an adequate Factor of Safety. This is the *Stable Foundation Zone*.

4.2.2 Piled and Piered Foundations

Structures may be founded on piles in such a way that the load is transmitted to strata below any possible failure zone within the dune. In this way a structure could be located within the *Zone Of Slope Adjustment*, albeit outside the zone of wave impact loading, and could, therefore, be located above the soil mass that has a factor of safety against slumping generally less than 1.5. For a soil to develop its shear strength the strain required along the failure plane is far greater than the strain required to shear a pile. Therefore, should a long pile intersect a possible failure plane, that is, should it be located partly within the soil mass for which the Factor of Safety is less than 1.0 (*Zone Of Slope Adjustment*), consideration will need to be given to the horizontal loads that may be applied to the pile should the slope slump and soil arching between piles occur. In the *Zone of Reduced Foundation Capacity* piers founded to a depth into the *Stable Foundation Zone* would not need to be designed for horizontal soil loading.

4.2.3 Seawalls

That a foreshore slope is lined with a seawall will not necessarily ensure its stability during extreme conditions. Vertical and non-porous seawalls are vulnerable and subject to sudden collapse should toe scour be coupled with a build up of water pressure behind the wall. Such walls are likely to experience scour at their toes additional to what otherwise may occur, thereby reducing further the stability of the slope.



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Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development
**Schema for Pile in
Zone of Slope Adjustment**

**Figure
4.2**

The zone of possible slip failures is likely to extend beyond and below any seawall lining of a frontal dune face. To optimise the stability of protected foreshores any seawall should be designed to be permeable and constructed at a relatively flat slope, particularly if development is to be allowed close to the crest.

4.3 Foundations in the Zone of Slope Adjustment

Pile foundations should be used in this zone and should be designed to support the following loads:

- axial loads transmitted from the structure;
- lateral loads transmitted from the structure; and
- lateral loads developed by slumping of the soil past the piles.

4.3.1 Design for Structural Axial Loads

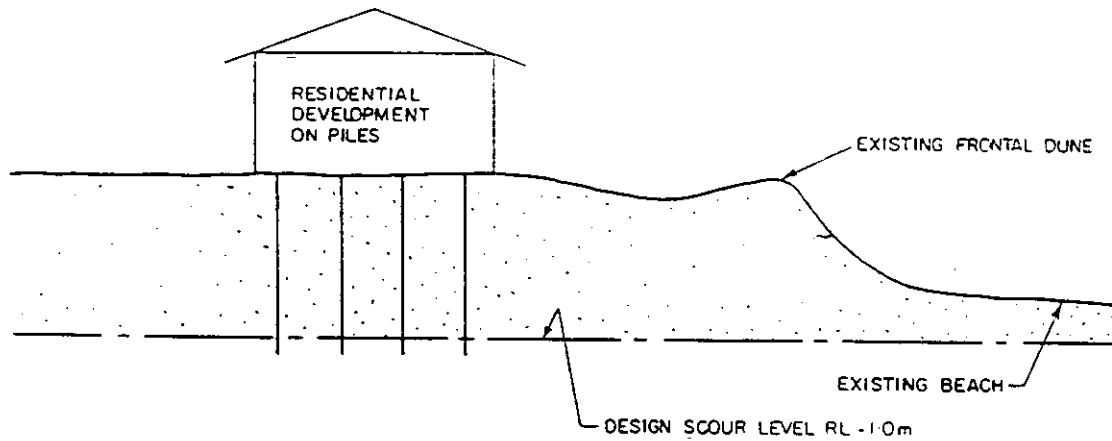
The piles should be designed in the conventional manner to satisfy the requirements of:

- an adequate factor of safety against geotechnical failure (i.e. failure of the supporting soil);
- an adequate factor of safety against structural failure of the piles themselves; and
- acceptable settlements under the design load.

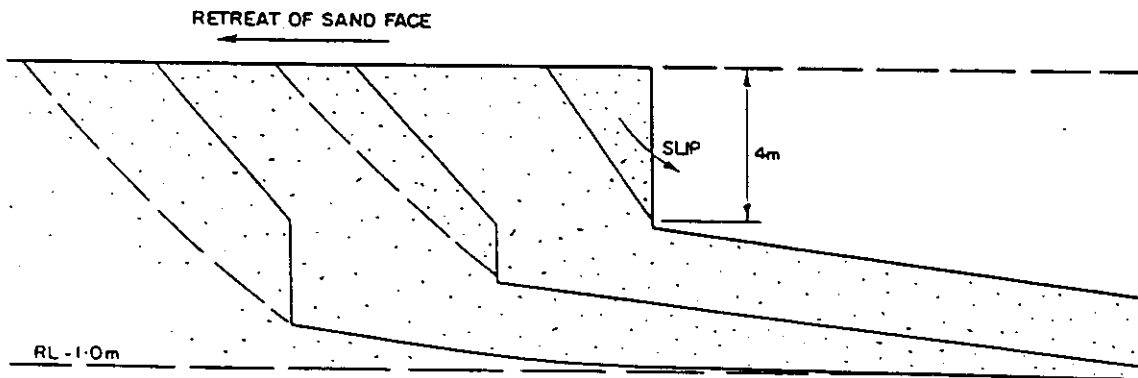
The Australian Standard Piling Code, AS2159-1978 (which is presently under revision), provides a basis for design for both load capacity and settlement. The required geotechnical parameters are:

- skin friction (shaft resistance) f_s ;
- end bearing capacity f_b ; and
- soil Young's modulus E_s .

Methods of assessing these parameters and of calculating axial capacity and settlement are given in the Piling Code. However, in assessing the geotechnical capacity of the pile, only that portion of the pile below scour level should be assumed to be effective in resisting the applied structural loads (see **Figure 4.2**). Similarly, when assessing the settlement of the pile, the settlement of the pile head should be taken as the sum of the compression of that portion of the pile above scour level, and the settlement of that portion of the pile below scour level.



DESIGN PROBLEM



Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development

**Soil Failure Schema
Adopted for Pile Design**

GEOMARINE

**Figure
4.3**

COFFEY

4.3.2 Design for Structural Lateral Loads

As with structural axial loads, the design of piles for structural lateral loads requires that the following criteria be satisfied:

- an adequate factor of safety against geotechnical failure of the piles;
- an adequate factor of safety against structural failure of the piles; and
- acceptable lateral deflections under the design loads.

The Australian Standard Piling Code again provides suitable approaches for the design of piles subjected to lateral loading. When applying these approaches, lateral resistance of the soil should only be assumed to exist below scour level.

4.3.3 Design for Soil Slumping Past Piles

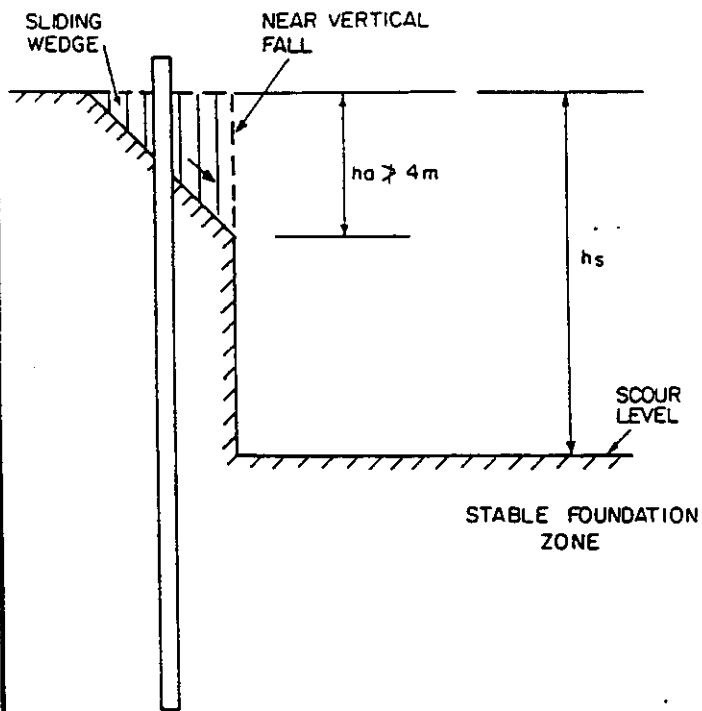
There are no well established criteria for this aspect of design, and therefore the treatment in this section is more extensive than for the design of piles subjected to axial or lateral structural loadings.

As with structural loadings, the design of piles to withstand soil slumping must consider:

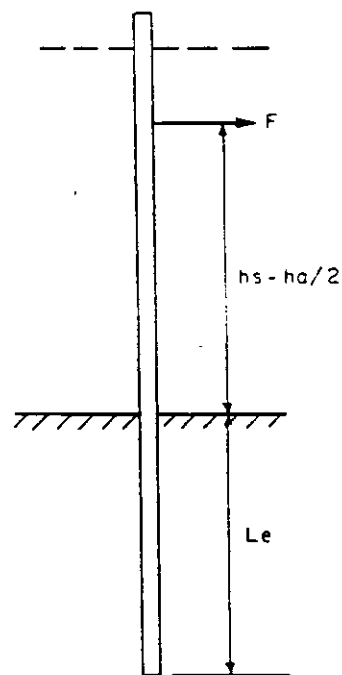
- the lateral geotechnical capacity of the piles;
- the lateral structural capacity of the piles; and
- the lateral deflection of the pile caused by the slumping.

Assessment of the geotechnical and structural capacities of the piles requires an estimate of the forces and bending moments induced in a pile by slumping of the soil past the pile.

It is postulated that the mechanisms of collapse of an eroded sand face retreating beneath a piled structure are as shown in Figure 4.3. It is assumed that a vertical face can form and then slump past the piles. Based on fundamental principles of soil mechanics, equations have been developed for lateral loads on piles due to wedge failure of sand behind a retreating vertical face. These equations are presented in Appendix A. A key feature is that the maximum bending moment depends on the cube of the vertical face height. There is, however, a limit to the vertical face height which can exist and, if it is assumed that a maximum suction of 40 to 50kPa can be sustained within the sand (over a short time interval when the sand is wet), then it is found that the maximum height of the vertical sand face that can be sustained is about 4m.



ASSUMED MECHANISM OF SOIL
SLUMPING PAST PILE



MODEL FOR DESIGN PILE TO
RESIST SLUMPING

Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development

Soil Loading Criteria for Pile Design

GEOMARINE

Figure
4.4

COFFEY

It is recommended that a maximum vertical face height of 4m be adopted for design. The resulting mechanism of soil slumping past the pile is illustrated in **Figure 4.4**, together with the model of the pile for design to withstand the slumping forces.

For practical calculations, it is further assumed that:

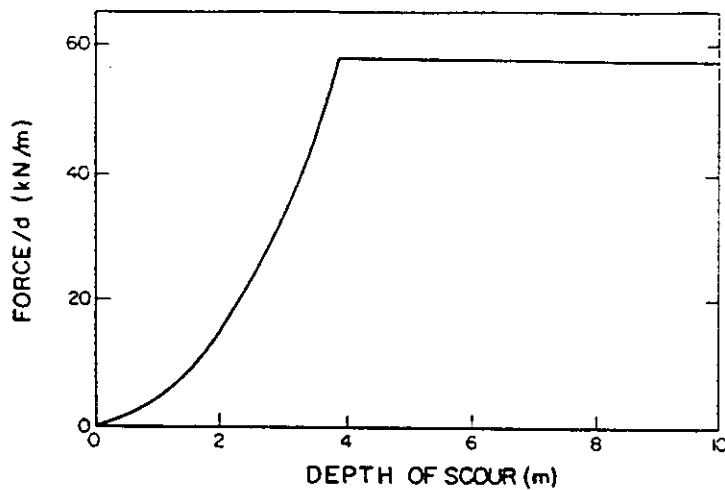
- the vertical face, after it slumps, comes to rest (at least temporarily) at a slope angle of 45° ; and
- each pile is acted upon by a wedge with a maximum width $B = 3d$, where d = pile diameter or width, unless the piles are spaced more closely than $3d$ centre-to-centre, in which case B = the centre-to-centre spacing between the piles.

Making these assumptions and adopting typical parameters for the slumping sand (assumed to be in a relatively loose condition) of friction angle $\phi = 30^\circ$ and unit weight $\gamma = 18 \text{ kN/m}^3$, the computed force and bending moment in the pile are shown in **Figure 4.5** as a function of the depth of scour. Because of the assumption made regarding a 4m maximum slump height, the force on the pile reaches a maximum value for scour heights equal to or greater than 4m (see **Figure 4.5a**); however, the bending moments continue to increase linearly with increasing scour depth, because of the increasing moment arm.

Figure 4.5b gives two curves:

- the bending moment at scour level (i.e. at the top of the supporting soil); and
- the maximum bending moment in the pile; this occurs in the portion of the pile embedded in the stable soil below scour level (it is assumed here that full soil resistance is mobilised above the point of maximum moment).

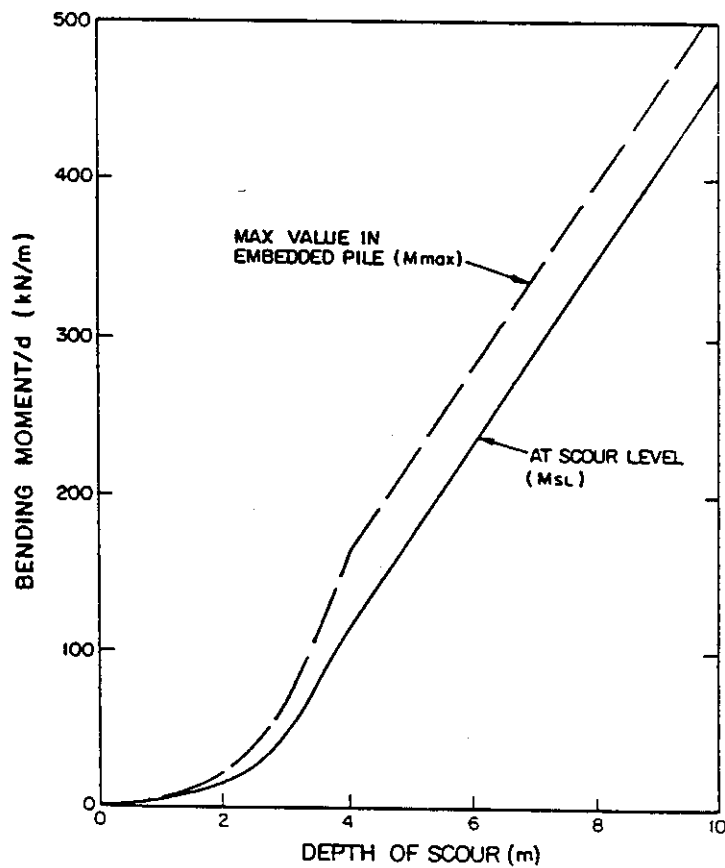
The curves in **Figure 4.5** provide the basis for design of the pile. It is considered reasonable to assume that, if the sand slumps past the pile, it will be in a relatively loose condition. However, if the friction angle of the soil or the unit weight differ markedly from the values adopted for **Figure 4.5**, the force and bending moment in the pile should be computed from the equations in **Appendix A**, bearing in mind the assumptions outlined above. **Figure A2** in **Appendix A** shows the effect of soil condition on the maximum shear force and bending moment at scour level. Assuming that B remains equal to 3 pile diameters, both the shear and bending moment decrease as the sand becomes more dense. Hence, the assumption of a loose sand in design will tend to be conservative.



DESIGN PARAMETERS

$\phi = 30^\circ$
 $\beta = 45^\circ$
 $\gamma = 19 \text{ kN/m}^3$
 $B = 3c$

a) SHEAR FORCE



b) BENDING MOMENTS

Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
**Shear Force and Bending Moments in
 Pile due to Slumping Sand Wedge**

GEOMARINE

**Figure
 4.5**

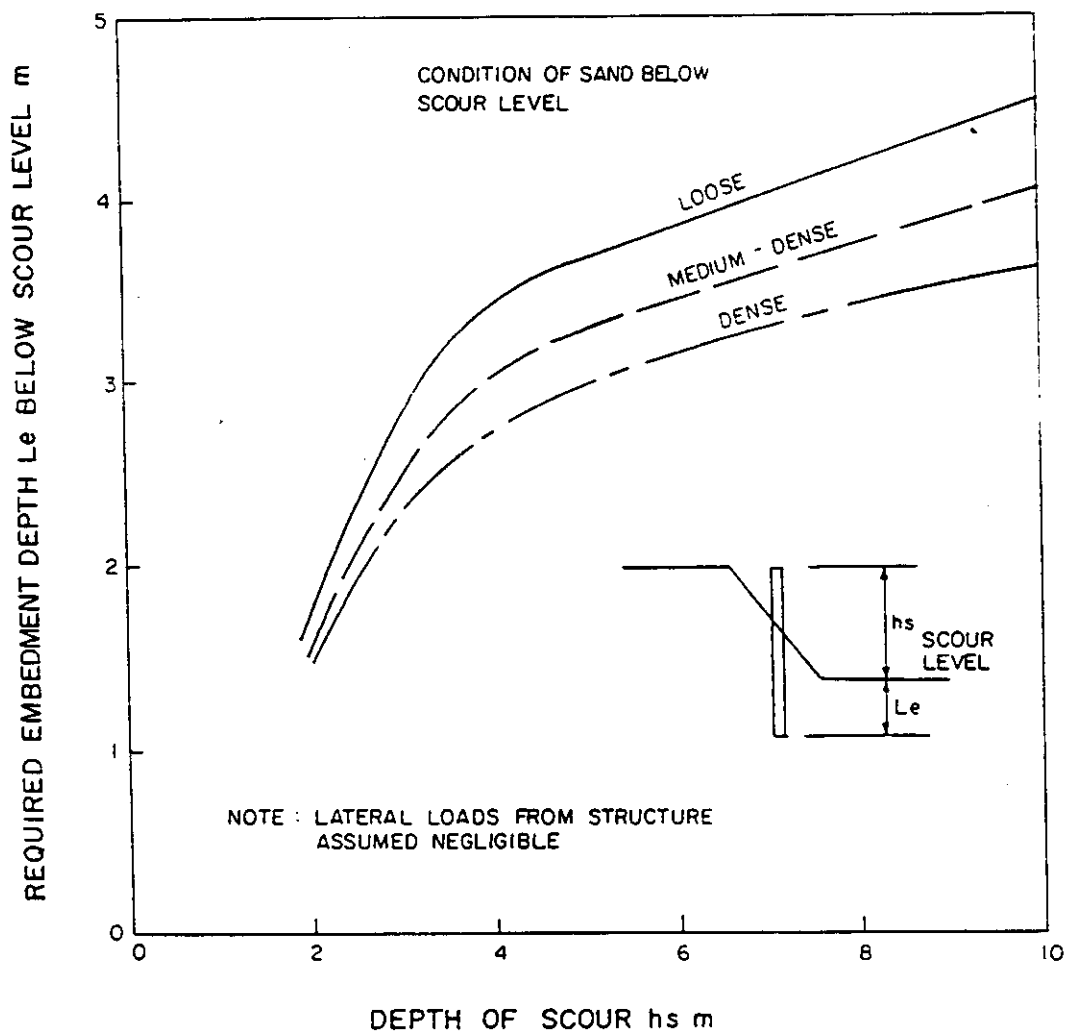
COFFEY

For geotechnical design, the pile must be embedded to a sufficient depth to resist the shear force and bending moment applied to the pile at scour level. Because of the likely extreme nature of the scouring process, it is considered that a *Factor Of Safety* of 1.5 should be adequate in this case. Using conventional lateral bearing capacity theory for piles (e.g. as presented in the Australian Standard Piling Code), it is possible to compute the required depth of embedment to develop a *Factor Of Safety* of 1.5 against lateral geotechnical failure. **Figure 4.6** plots the required embedment depth so computed, as a function of the depth of scour, h_s . Curves are shown for three sand conditions below scour level (in all cases the supporting sand is assumed saturated):

- loose sand, for which a friction angle ϕ of 30° and an effective unit weight of $\gamma = 18\text{kN/m}^3$ are assumed;
- medium dense sand (assumed $\phi = 35^\circ$, $\gamma = 19\text{kN/m}^3$);
- dense sand (assumed $\phi = 38^\circ$, $\gamma = 20\text{kN/m}^3$).

As would be expected, the required depth of embedment L_e decreases as the sand becomes more dense. However, for most practical circumstances, L_e appears to lie within the range of 2.5 to 4.5m. It should, however, be emphasised that this required embedment depth is for the lateral forces due to soil slumping only; consideration must be given also to the embedment requirements for structural axial and lateral loadings.

For structural design of the pile, the pile section should be designed to withstand the maximum bending moment M_{\max} as shown in **Figure 4.5**, with an appropriate margin of safety. For assessment of lateral deflections due to the slumping of the slope, the model shown in **Figure 4.4** can be used, and conventional methods of calculation (e.g. as in the Australian Standard Piling Code) employed. The lateral deflection and rotation of the pile at the scour level are computed by assuming the pile to be subjected to the lateral force and bending moment (M_{\max}) shown in **Figures 4.5a** and **4.5b** respectively. The additional deflection above the scour level (due to rotation and bending of the free-standing portion of the pile) is then computed and added to the deflection of scour level to obtain the lateral deflection of the structure. This value does not represent a permanent deflection, and will be at least partially recovered when the sand face retreats past the pile.



Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
**Required Embedment Depth of Piles
 in Zone of Slope Adjustment**

GEOMARINE

Figure
 4.6

COFFEY

It should be noted also that the slumping of the slope will tend to induce an additional axial force in the pile due to the effects of negative friction. However, because of the values of negative skin friction near the top of the pile are likely to be low in sands, it is found that this additional axial force is generally negligible (typically of the order of 2 kN or less).

4.4 Foundations in the Zone of Reduced Foundation Capacity

In this zone, either shallow or deep foundations can be used, and should be designed to support the structural axial and lateral loadings. As before, the questions of geotechnical capacity, structural capacity and deformations under the design loads must be considered. Conventional methods of foundation analysis may be used; however, allowance must be made in the assessment of the geotechnical capacity of the foundation for the reduced resistance due to the proximity of the slumped soil escarpment. In the absence of adequate theoretical procedures for making such assessments, it is recommended that the required load capacity of the foundation be developed within the underlying Stable Foundation Zone, and that no allowance be made for the resistance of the soil in the soil above this zone. Thus, referring to Figure 4.7, for a spread footing, the base of the footing should be completely within the stable foundation zone, and it is assumed that no resistance is developed in the soil above the line A-A. For a pile foundation, no resistance is assumed above line B-B, so that the required axial and lateral capacity must be developed in the stable zone below this level. These assumptions are likely to be conservative, but it is believed that they provide a reasonable basis for foundation design, given the present state of knowledge. With these assumptions, the foundations in the zone may be designed by conventional foundation engineering principles. The parameters required for design are discussed below.

4.5 Geotechnical Parameters Required for Design

A summary of the main geotechnical parameters required for foundation design in coastal and beach developments is given in Table 4.1. Practical methods of assessing these parameters are discussed below.

Parameter; Definition

Uses

f_{all} ; allowable bearing pressure

estimation of size of shallow foundation

f_s ; pile skin friction

calculation of ultimate shaft capacity of pile

f_b ; pile end bearing capacity

calculation of ultimate end bearing capacity of pile

E_s ; Young's modulus of soil

calculation of foundation movements; different values are generally applicable for shallow and pile foundations, and for axial and lateral movements

γ ; unit weight

ϕ ; friction angle

estimation of ultimate lateral capacity of pile and assessment of additional lateral force and moment developed in pile by slumping soil

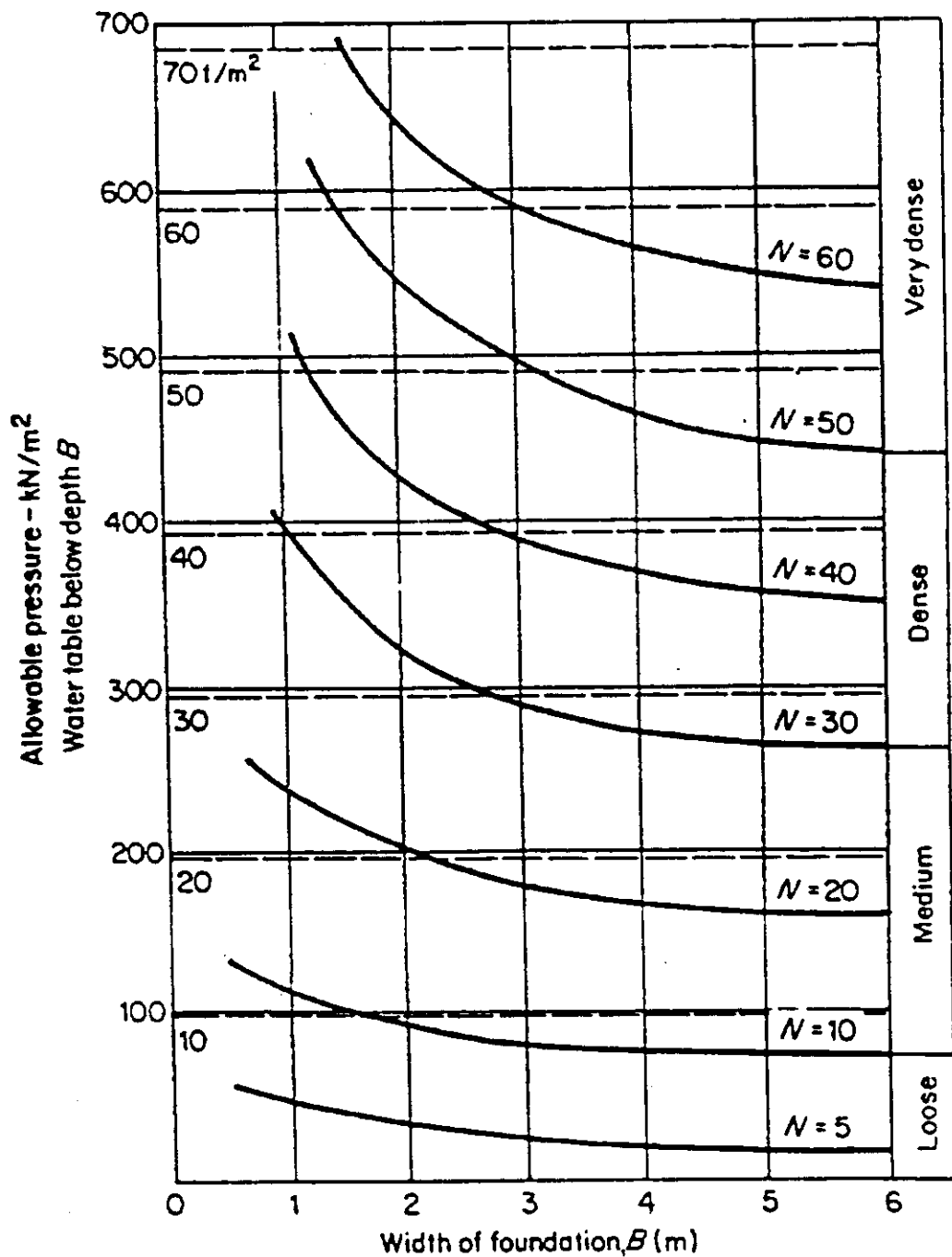
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Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development

**Summary of Main Geotechnical
Parameters Required**

Table

4.1



Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Allowable Bearing Pressures for Foundations
 in Sands Based on SPT Results

COFFEY

Figure
 4.8

GEOMARINE

Values of F, f_{sl} , N_q and f_{bl} for Piles in Silica Sand

Condition of Soil	Relative Density	Skin Resistance				Ultimate Base Resistance			
		Displacement Piles (1)		Non Displacement Piles (2)		Displacement Piles		Non Displacement Piles	
		F	f_{sl} kPa'	F	f_{sl} kPa	N_q	f_{bl} MPa	N_q	f_{bl} MPa
Loose	0.2-0.3	0.8	25	0.3	10	60	2.0	25	0.8
Medium-loose	0.3-0.4	0.9	35	0.4	16	75	3.0	40	1.6
Medium	0.4-0.6	1.0	50	0.5	25	100	5.0	60	3.0
Medium-dense	0.6-0.75	1.2	65	0.65	35	130	7.0	80	4.2
Dense	0.75-0.9	1.5	85	0.8	45	180	10.0	100	5.5
Very dense	>0.9	1.75	100	0.9	50	210	12.0	120	7.0

1. Including cast in place piles of hammered shaft construction.
2. Assuming close supervision of construction is exercised.

Warringah Shire Council

Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
Values of F, f_{sl} , N_q and f_{bl} for Piles in Silica Sand

**Table
4.2**

$$f_s = F\sigma_v' \leq f_{sl} \quad \dots (2)$$

where:

$$\begin{aligned} \sigma_v' &= \text{vertical effective stress;} \\ F &= \text{skin friction factor; and} \\ f_{sl} &= \text{limiting value of skin resistance.} \end{aligned}$$

Values of F and f_{sl} are shown in Table 4.2 for normal silica-based sands. f_s may also be correlated approximately with standard penetration test (SPT) data. For piles in silica sand, Meyerhof (1956) suggests the following correlation for the average skin friction f_s along the pile shaft:

$$f_s = 2C_T \bar{N} \text{ kPa} \quad \dots (3)$$

where:

$$\begin{aligned} \bar{N} &= \text{average SPT value along shaft;} \\ C_T &= \begin{aligned} &1.0 \text{ for large displacement piles; and} \\ &0.5 \text{ for small displacement piles and for} \\ &\text{bored piles.} \end{aligned} \end{aligned}$$

Alternative correlations with static cone penetration test data are summarised in Poulos (1989) and are shown in Figure 4.9.

The above values are used to estimate the ultimate pile shaft capacity by integrating the skin friction over the surface area of the pile shaft. A suitable factor of safety, typically 2.5, needs to be applied to the ultimate value to obtain an allowable value of shaft capacity.

4.5.3 Pile End Bearing Resistance f_b

The draft revised Australian Piling Code suggests that f_b can be estimated as follows:

$$f_b = \sigma_{vb}' N_q \leq f_{bl} \quad \dots (4)$$

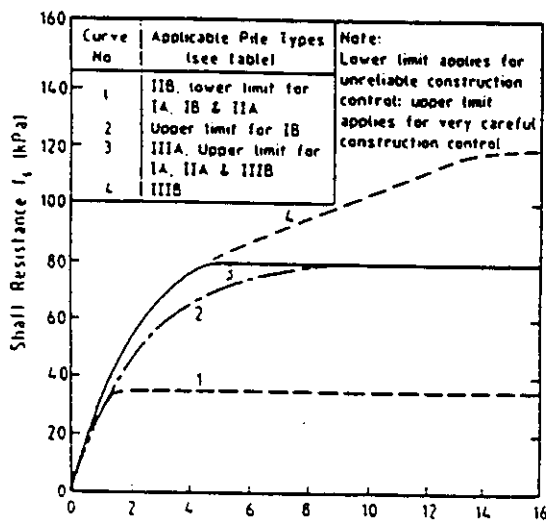
where:

$$\begin{aligned} \sigma_{vb}' &= \text{effective vertical overburden pressure at} \\ &\text{level of pile tip;} \\ N_q &= \text{bearing capacity factor; and} \\ f_{bl} &= \text{limiting value of end bearing resistance.} \end{aligned}$$

Recommended values of N_q and f_{bl} are shown in Table 4.2.

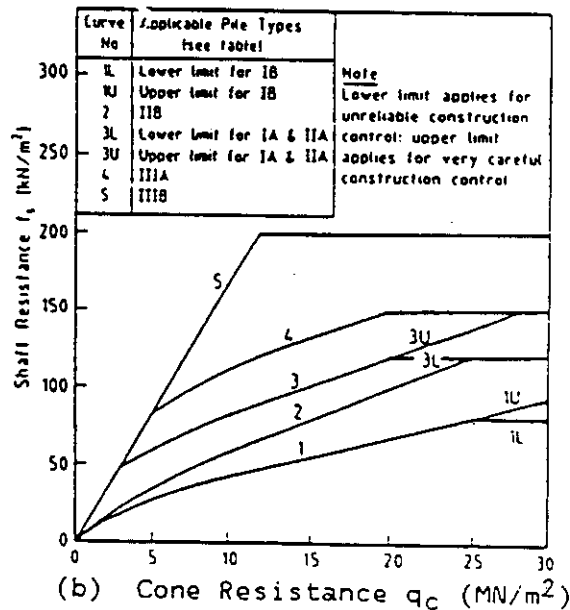
Pile Category	Type of pile
IA	Plain bored piles, mud bored piles, hollow auger bored piles, cast screwed piles
IB	Cased bored piles Driven cast piles
IIA	Driven precast piles Prestressed tubular piles Jacked concrete piles
IIB	Driven steel piles Jacked steel piles
IIIA	Driven grouted piles Driven rammed piles
IIIB	High pressure grouted piles ($d > 0.25$ m) Type II micropiles

CLAYS



(a) Cone resistance q_c (MPa)

SANDS



(b) Cone Resistance q_c (MN/m^2)

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Foundation Design Criteria for Residential Development
**Correlations Between Ultimate Shaft
Resistance and Static Cone Resistance**

GEOMARINE

Figure
4.9

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Correlations between f_b and SPT data are summarised in Poulos (1989), and can be expressed as:

$$f_b = KN \text{ MPa} \quad \dots (5)$$

where:

- N = average SPT value in vicinity of pile tip; and
 K = factor depending on pile and soil type.

K ranges from about 0.4 for driven piles in sand to 0.1 for bored piles in sand.

f_b can be roughly correlated also with static cone penetration resistance data:

$$f_b = K_1 q_c \quad \dots (6)$$

where:

- q_c = average cone resistance in vicinity of pile tip;
 K_1 = factor which generally lies between 0.1 and 0.55, depending on soil and pile type.

The above values are used to estimate the ultimate end bearing capacity of the pile by multiplying by the area of the pile base. A suitable factor of safety, typically 2.5, needs to be applied to the ultimate value to obtain an allowable value of pile base load.

4.5.4 Young's Modulus E_s

Because soil is not an elastic material, the equivalent value of Young's modulus to be used for foundation deformation calculations will depend on the type of foundation and the type of loading, as well as the type of soil. Different methods of calculation may also require different correlations to be employed.

4.5.4.1 Shallow Foundations

One of the procedures more commonly used for estimating settlements of shallow foundations on sands is that proposed by Schmertmann (1970) and summarised in Tomlinson (1986). In this method, E_s is correlated with the static cone resistance q_c as follows:

$$E_s = \alpha q_c \quad \dots (7)$$

where:

$$\alpha = \begin{array}{l} 2.5 \text{ for circular and square foundations; and} \\ 3.5 \text{ for strip or long (length/breadth} > 10) \\ \text{rectangular foundations, and is interpolated} \\ \text{between the above values for rectangular} \\ \text{foundations.} \end{array}$$

An alternative approach for settlement estimation, based on SPT data, has been proposed by **Burland and Burbridge (1985)** and is summarised also in **Tomlinson (1986)**.

4.5.4.2 Pile Foundations

(a) Axial Loading

For axially loaded piles in sand the Australian Standard Piling Code suggests the average values of E_s shown in Table 4.3 for driven piles. Some correlations between E_s and cone penetration resistance are summarised in **Poulos (1989)**.

Table 4.3 Typical Values of Young's Modulus for Driven Axially Loaded Piles in Silica Sand

Soil Type	E_s (MPa)
Loose sand	42
Medium sand	70
Dense sand	90
Very dense sand	200

(b) Lateral Loading

For laterally loaded piles in sand, it is frequently assumed that Young's modulus increases linearly with depth, so that:

$$E_s = C_2 N_h Z \quad \dots (8)$$

where:

$$C_2 = \begin{array}{l} 1.0 \text{ for dry and moist sands; and} \\ 0.66 \text{ for saturated sands.} \end{array}$$

Values of N_h suggested by the Australian Standard Piling Code are shown in Table 4.4. Alternative correlations for this case have been proposed by **Kishida and Nakai (1977)** who suggest:

$$E_s = 1.6N \text{ MPa} \quad \dots (9)$$

where:

$$N = \text{SPT value.}$$

Table 4.4 Typical Values of Soil Modulus Gradient for Laterally Loaded Piles in Sand

Soil Type	N_h (MPa/m)
Loose sand	1.6
Medium dense sand	4.8
Dense sand	12.6

4.5.5 Unit Weight, γ

The unit weight of a soil is related to its void ratio, e , and the unit weight of water γ_w and will be in the range between the dry unit weight γ_d and the saturated unit weight γ_{sat} , where:

$$\gamma_d = G\gamma_w/(1 + e) \quad \dots(10)$$

$$\gamma_{sat} = (G + e)\gamma_w/(1 + e) \quad \dots(11)$$

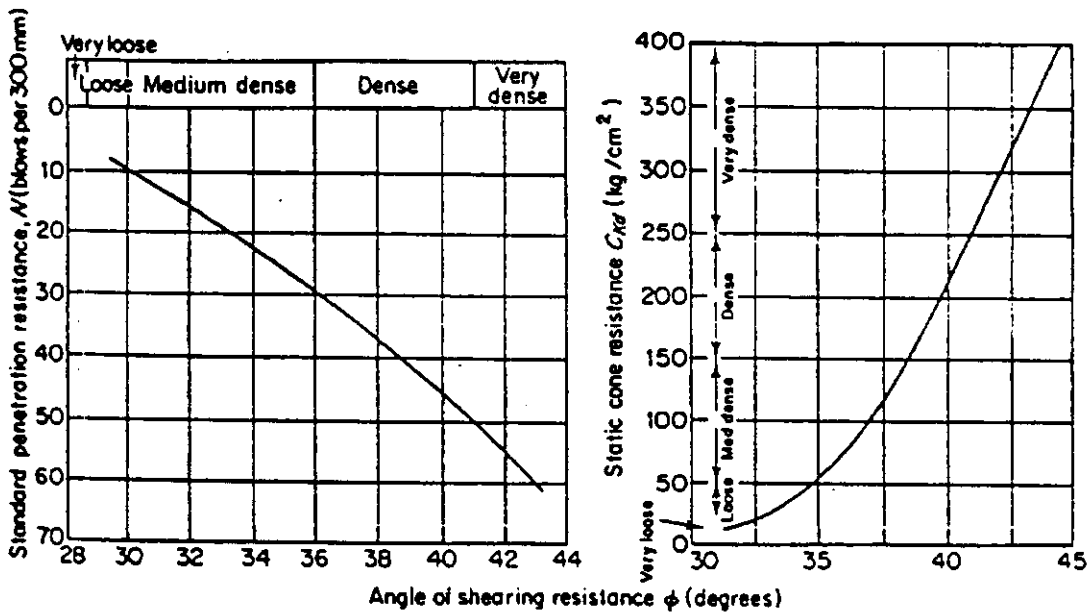
where:

$$G = \text{specific gravity of soil particles, typically 2.68 for quartz sands}$$

When a site investigation has established the level of the water and enabled the void ratio to be estimated, the value of γ can readily be calculated. For common ranges of void ratios in sands (between 0.5 and 0.9), γ will lie between 13.4 and 20.8 kN/m³, although more commonly the range is narrower, between 16 and 20 kN/m³.

4.5.6 Angle of Friction ϕ

The angle of friction of a sandy soil may be established by means of laboratory testing, the most convenient test for this purpose being a direct shear test. Correlations have also been suggested between ϕ and the results of conventional insitu tests, and two such correlations are shown in **Figure 4.10** (Tomlinson, 1986). ϕ may be influenced also by the shape of the sand grains and tends to be lower for sands with rounded grains than for those with angular grains.



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Determination of ϕ from In-situ Tests

Figure
4.10

5. Typical Foundation Conditions and Requirements for the Collaroy-Narrabeen-Fishermans Beach Areas

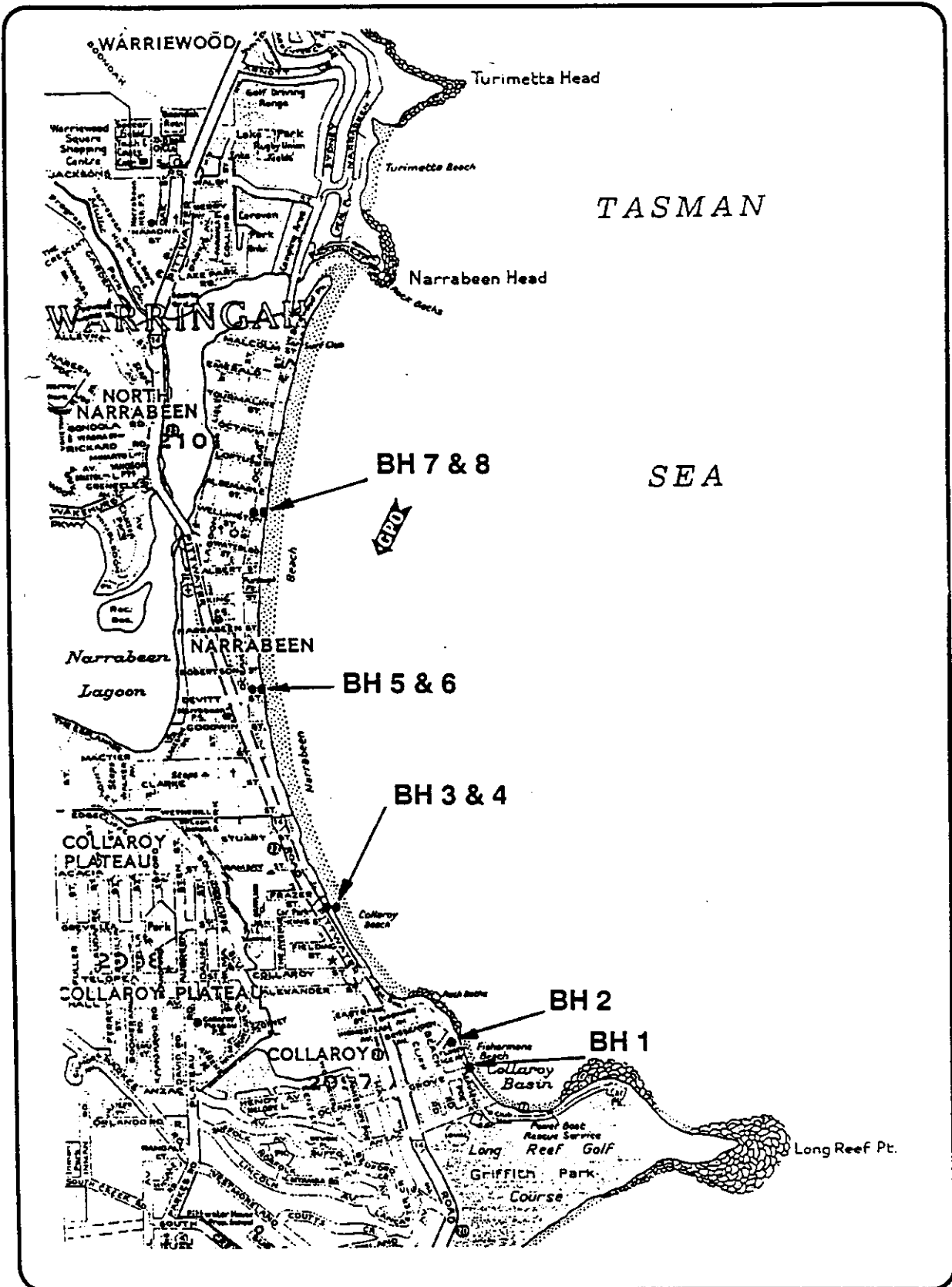
5.1 Introduction

In this section the design procedure is used with appropriate geotechnical parameters for the Collaroy-Narrabeen-Fishermans Beach areas to develop typical foundation design requirements. However, it is neither possible nor desirable to develop "standard" foundation designs as the geotechnical conditions and structural loads vary from project to project.

5.2 Results of Investigation and Assessment of Design Conditions

Field investigations at specific sites were carried out between 30th November and 4th December 1990. Field work comprised auger drilling together with electric friction cone testing. Eight holes were drilled to maximum depths of 10m at locations shown in Figure 5.1.

The soil profile encountered in the boreholes is shown in the logs presented in **Appendix B**. The logs of boreholes 3 to 8 have been summarised into cross sections which are presented in **Appendix B**. The cross sections provide also a profile of SPT results and water level recordings.



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Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Borehole Locations

Figure
 5.1

BH No.	Location	Depth Range (m)	SPT (N)	Broad Classification
BH1	Fishermans Beach South end	0.0 to 1.3		Fill and topsoil
		1.3 to 2.9	5	Medium dense alluvial clayey sand
		2.9 to 5.7	13	Hard, high plasticity alluvial clay
		5.7 to 10.5	19	Hard, medium plasticity residual clay
BH2	Fishermans Beach North end	0.0 to 1.2		Fill
		1.2 to 3.7	2	Loose beach sand with shell fragments
		3.7 to 4.7	1	Very loose clayey sand
		4.7 to 5.7	R	EW-HW claystone
BH3	Collaroy Beach North end	0.0 to 0.3		Fill
		0.3 to 1.6	5	Medium dense dune sand
		1.6 to 3.7	31	Very dense dune sand
		3.7 to 5.0	41	Very dense cemented beach sand
		5.0 to 7.1	5	Loose beach sand
		7.1 to 8.6	3	Loose estuarine clayey sand
8.6 to 9.3	15	Stiff to very stiff estuarine clay		
BH4	Collaroy Beach North end	0.0 to 7.8	20-35	Dense to very dense dune sand
		7.8 to 9.3		Very dense estuarine clayey sand
		9.3 to 10.1		Estuarine sandy clay
BH5	Narrabeen Beach South end	0.0 to 0.8		Fill
		0.8 to 5.5	10-15	Medium dense to dense dune sand
		5.5 to 10.1	40	Very dense dune sand
BH6	Narrabeen Beach South end	0.0 to 0.9		Fill
		0.9 to 5.2	8-12	Medium dense to dense dune sand
		5.2 to 7.0	30	Dense dune sand
		7.0 to 10.25	R	Very dense indurated sand
BH7	Narrabeen Beach	0.0 to 1.1		Fill
		1.1 to 8.3	10-15	Medium dense dune sand
		8.3 to 10.2	R	Very dense indurated sand
BH8	Narrabeen Beach	0.0 to 5.3	10-15	Medium dense dune sand
		5.3 to 7.8	12->30	Medium to very dense dune sand
		7.8 to 10.25	>30	Very dense indurated sand

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Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development
Broad Geotechnical Parameters

see Figure 5.1 for borehole locations

COFFEY

Table 5.1

GEOMARINE

Boreholes 5 to 8 encountered a fairly uniform profile of medium grained sands containing little or no fines. Below the water table, these sands became cemented by calcareous and iron oxide cements. Boreholes 3 and 4 encountered a layer of sandy clay at about RL -1m which graded into a grey high plasticity clay at RL -3m in BH3. Boreholes 1 and 2 encountered dune sands overlying weathered shale and claystone.

Laboratory testing on selected samples consisted of particle size gradings and direct shear testing. The results of laboratory testing are presented in Appendix C.

Based on the results of the field investigations and the laboratory data our assessments of the broad geotechnical conditions at the various locations investigated are summarised in Table 5.1. At almost all locations, hard clay or dense sand layers are encountered at depths ranging between about 5 and 9m below surface level. Such layers appear to be very suitable for providing foundation support. Typical values of unit weight and angle of friction are shown in Table 5.2. As a first approximation, the foundation design parameters can then be estimated as follows:

(a) **Shallow Foundations**

f_{all} **Figure 4.8**
 E_s **Equation 7**

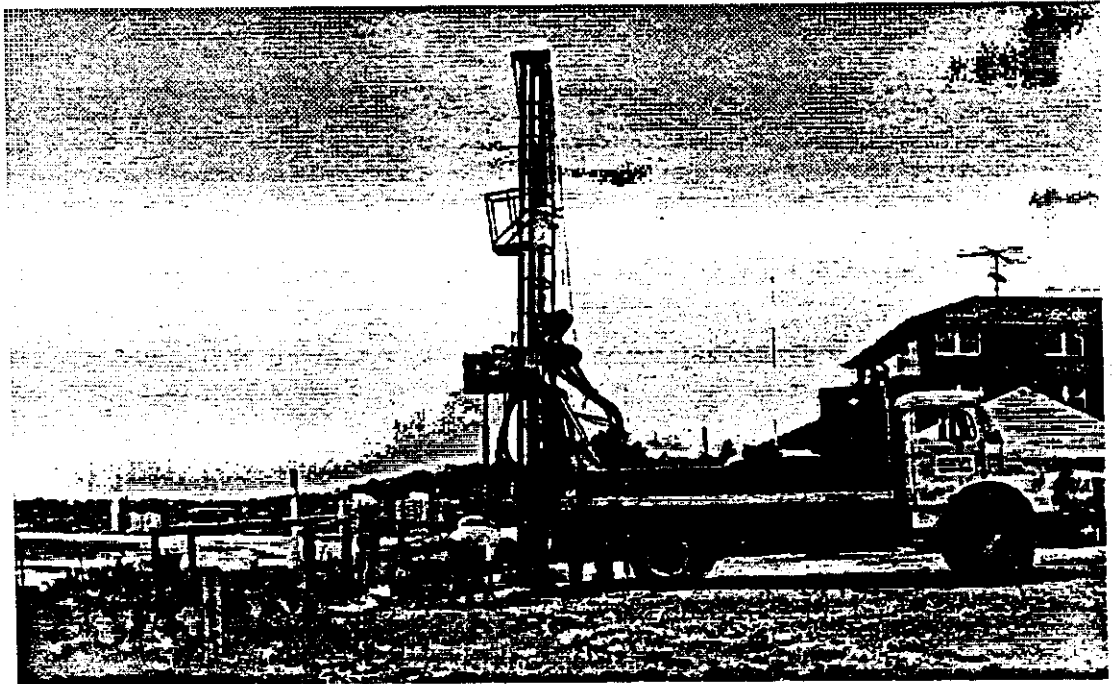
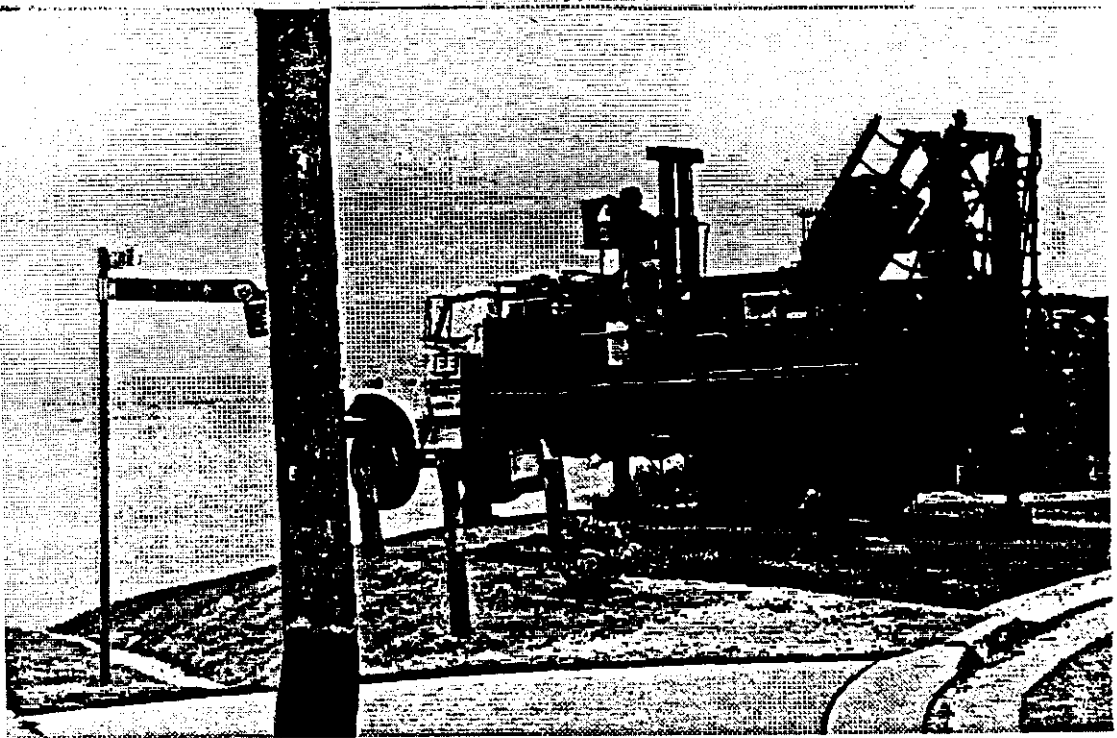
(b) **Pile Foundations**

f_s **Table 4.2, Equations 2 and 3**
 f_b **Table 4.2, Equations 4 to 6**
 E_s (axial loading) **Table 4.3**
 E_s (lateral loading) **Table 4.4, Equations 8 and 9**

Table 5.2 Typical Values of Unit Weight and Angle of Friction for the Collaroy-Narrabeen Beach Areas

Soil Type	Typical γ^* (kN/m ³)	Typical ϕ (degrees)
Loose sand	18	30
Medium dense sand	19	35
Dense Sand	20	38
Clay	18	N/A

*saturated unit weight



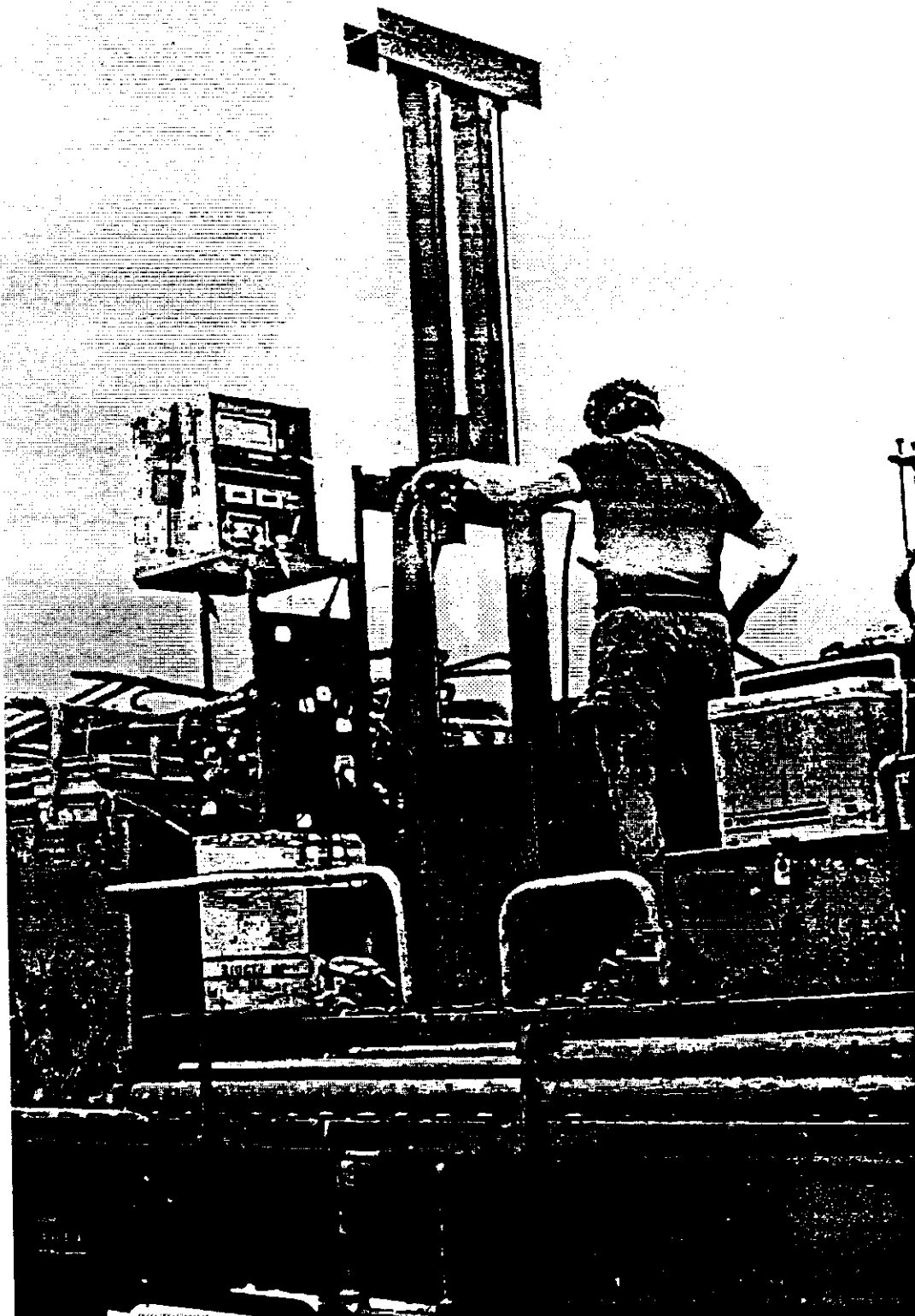
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**Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development
Foundation Investigations
November, 1990**

COFFEY

**Plate
5-1**

GEOMARINE



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**Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development
Detailed View of Dutch Cone
Penetrometer Rig**

GEOMARINE

**Plate
5.2**

COFFEY

5.3 Foundation Requirements

5.3.1 Zone of Slope Adjustment

In this zone only piles can be used for foundation support and such piles must be designed to withstand forces and moments induced in the pile by a slumping soil face as well as structural vertical and lateral loads.

The minimum depth of embedment of such piles into the underlying stable zone is indicated in **Figure 4.6**. For the depths of scour likely to be encountered in the Collaroy-Narrabeen area the required minimum embedment depth below scour level is likely to be between 3 and 4m. However, the pile must be checked also for axial and lateral structural loadings to ensure that the embedment depth is adequate for the effects of combined loading and that the pile section is not overstressed under the combined action of structural and soil loadings.

5.3.2 Zone of Reduced Foundation Capacity

In this zone, either piles or shallow foundations may be used to support the structural, vertical and lateral loads, but the resistance must be developed from the stable soil below the zone of reduced foundation capacity.

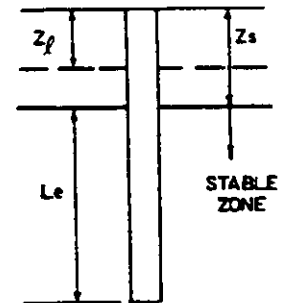
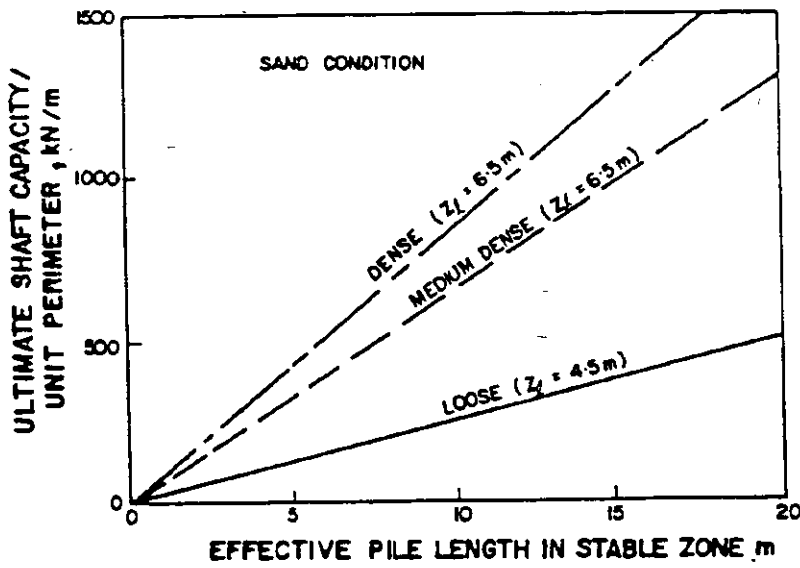
To provide some guidance on the axial load capacity and settlement of piles in the Collaroy-Narrabeen-Fishermans Beach areas the recommended design parameters have been used to prepare design charts which are shown in **Figures 5.2** and **5.3**.

Figure 5.2 shows the ultimate shaft capacity, per unit perimeter, as a function of effective pile length in the stable zone. When the depth from the soil surface to the stable zone, z_s , exceeds the depth, z_1 , at which the skin friction reaches its limiting value, the effective pile length is equal to the length, L_e , within the stable zone. If z_s is less than z_1 , the effective length is reduced as shown in **Figure 5.2** to allow for the fact that the design skin friction has not reached a constant value.

Figure 5.3 shows the ultimate end bearing capacity of a pile as a function of base diameter. In this figure it is assumed that the pile base lies below the depth z_1 required for the limiting end bearing pressure to be developed.

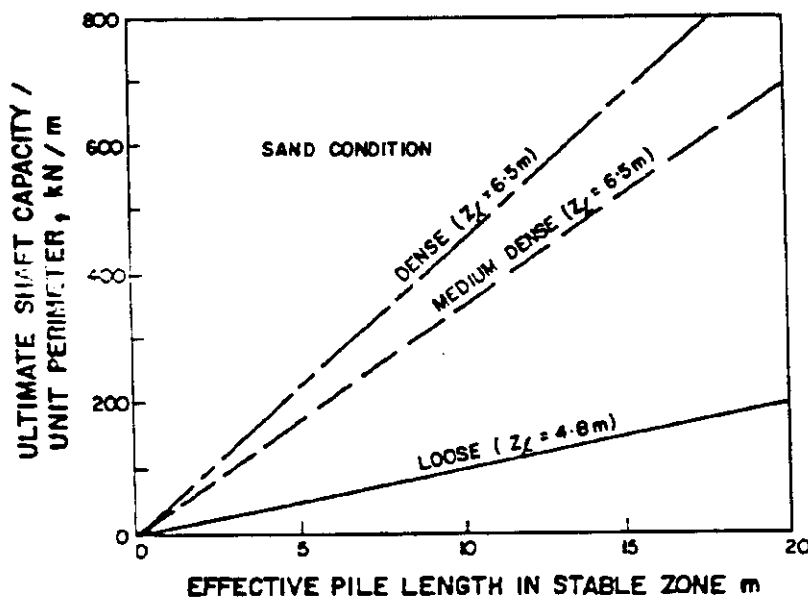
Figures 5.4 to 5.9 show the settlement per unit load for both driven and bores piles in sands of different densities. These charts have been prepared using typical design parameters and the approach described in **Poulos (1989)**.

The settlement referred to is for the portion of the pile within the stable zone. Group effects should be allowed for, for example, as suggested in **Poulos (1989)**. To this settlement must be added the compression of the pile above the stable layer, assuming that no load transfer occurs within the zone of reduced foundation capacity. A simple chart for estimating this compression is given in **Figure 5.10**, assuming that the equivalent Young's modulus of the pile section is 25000 MPa. This chart would be applicable to concrete piles and to steel tube piles with a steel area of about 12% of the gross cross-sectional area of the piles.



IF $Z_s > Z_f$
EFFECTIVE LENGTH = L_e
IF $Z_s < Z_f$
EFFECTIVE LENGTH = $L_e - \frac{(Z_f - Z_s)}{2}$

a) DISPLACEMENT PILES



b) NON-DISPLACEMENT PILES

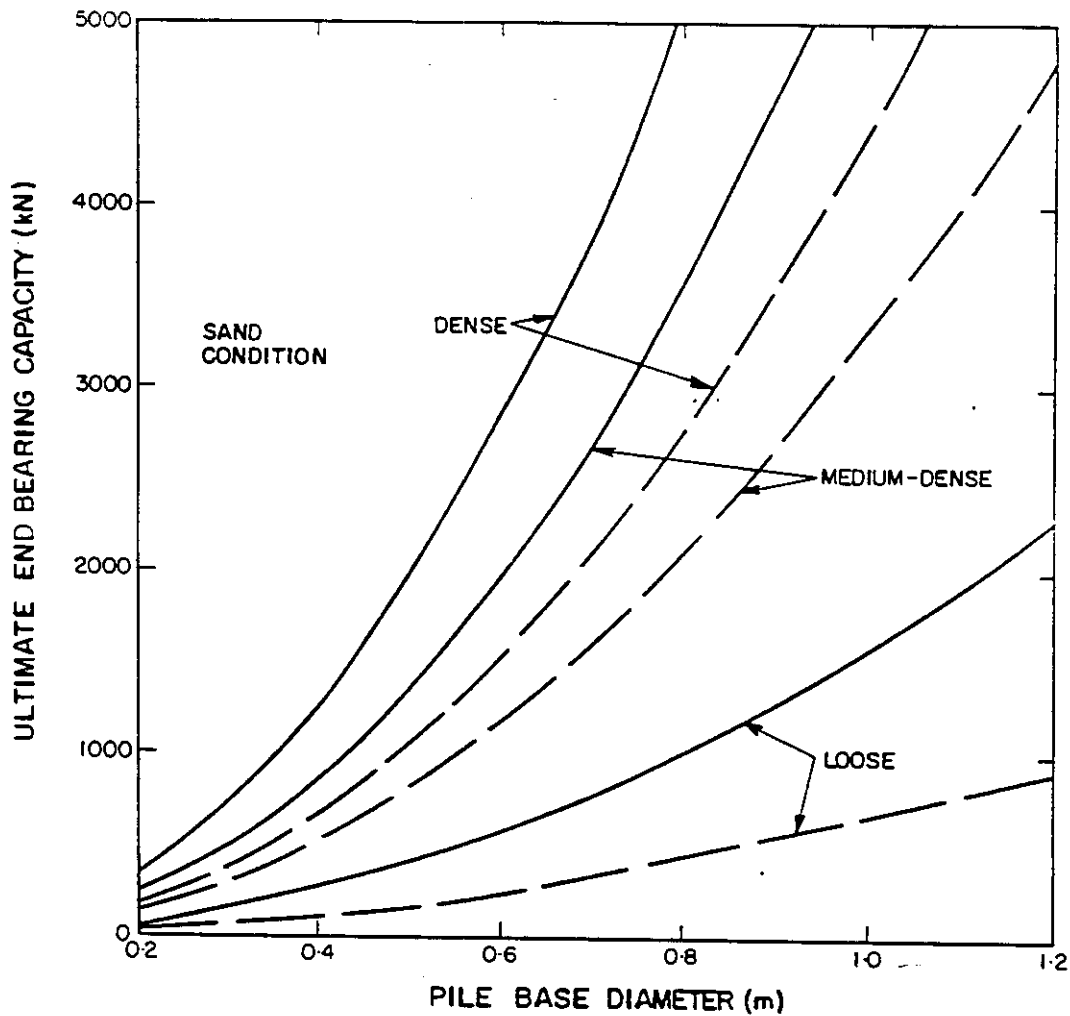
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Narrabeen-Collaroy-Fishermans Beach
Foundation Design Criteria for Residential Development
Design Charts for Ultimate Pile Shaft
Capacity in Sand

COFFEY

Figure
5.2

GEOMARINE



LEGEND
 ——— DISPLACEMENT PILES
 - - - NON-DISPLACEMENT PILES

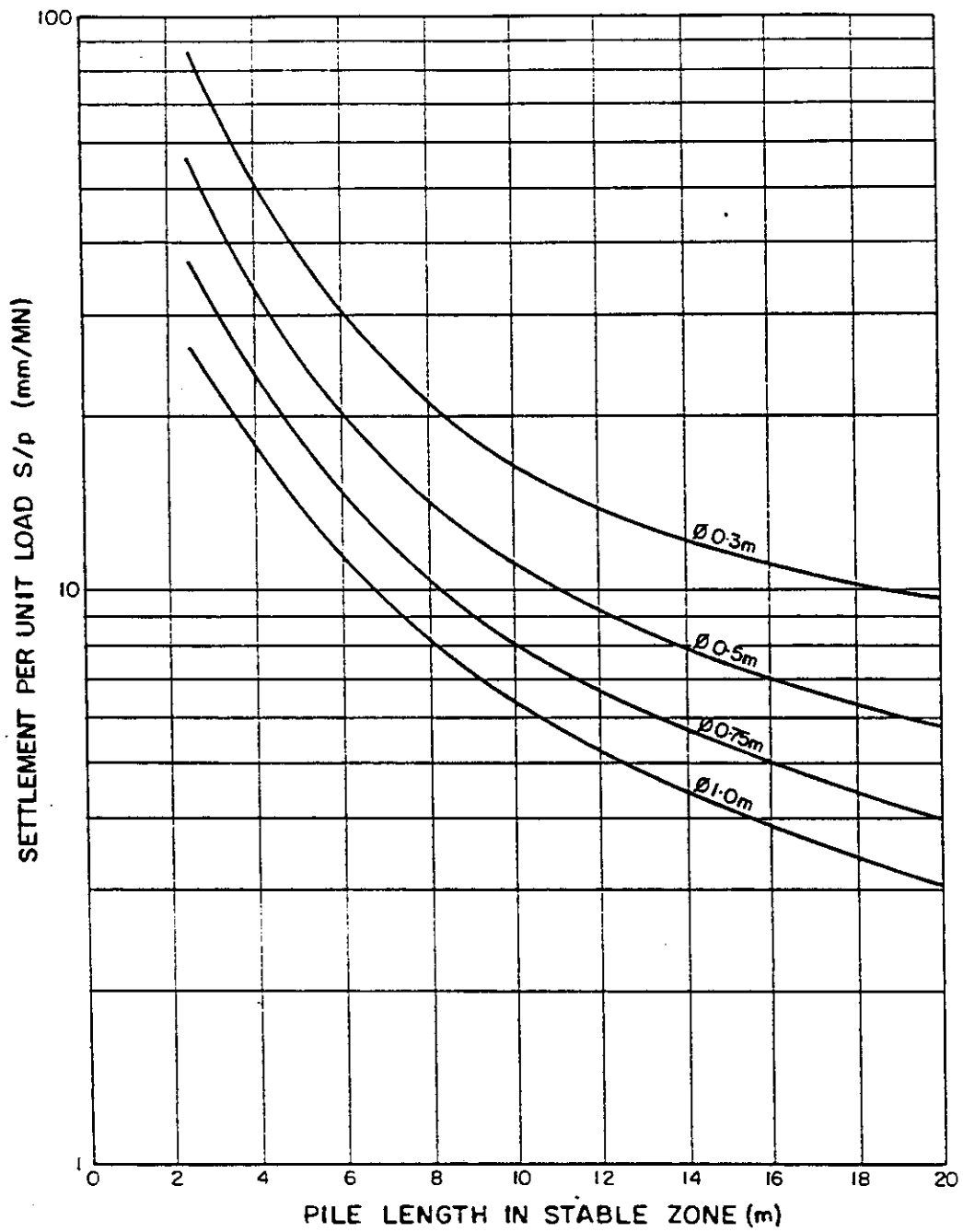
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 Foundation Design Criteria for Residential Development
**Design Chart for
 Ultimate Base Load Capacity in Sand**

GEOMARINE

**Figure
 5.3**

COFFEY



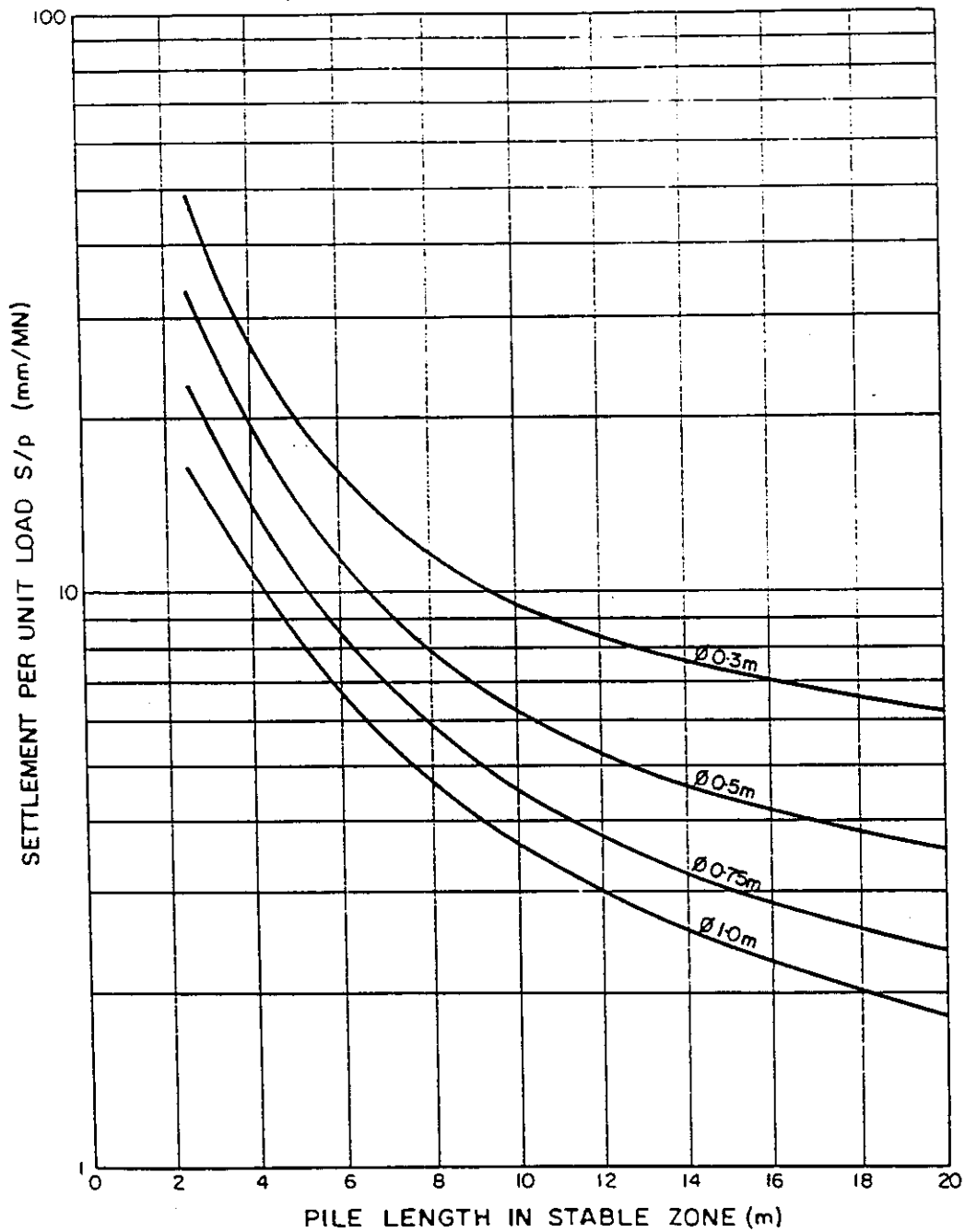
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 Foundation Design Criteria for Residential Development
 Design Chart for Settlement of Single
 Driven Pile in Loose Sand

COFFEY

Figure
 5.4

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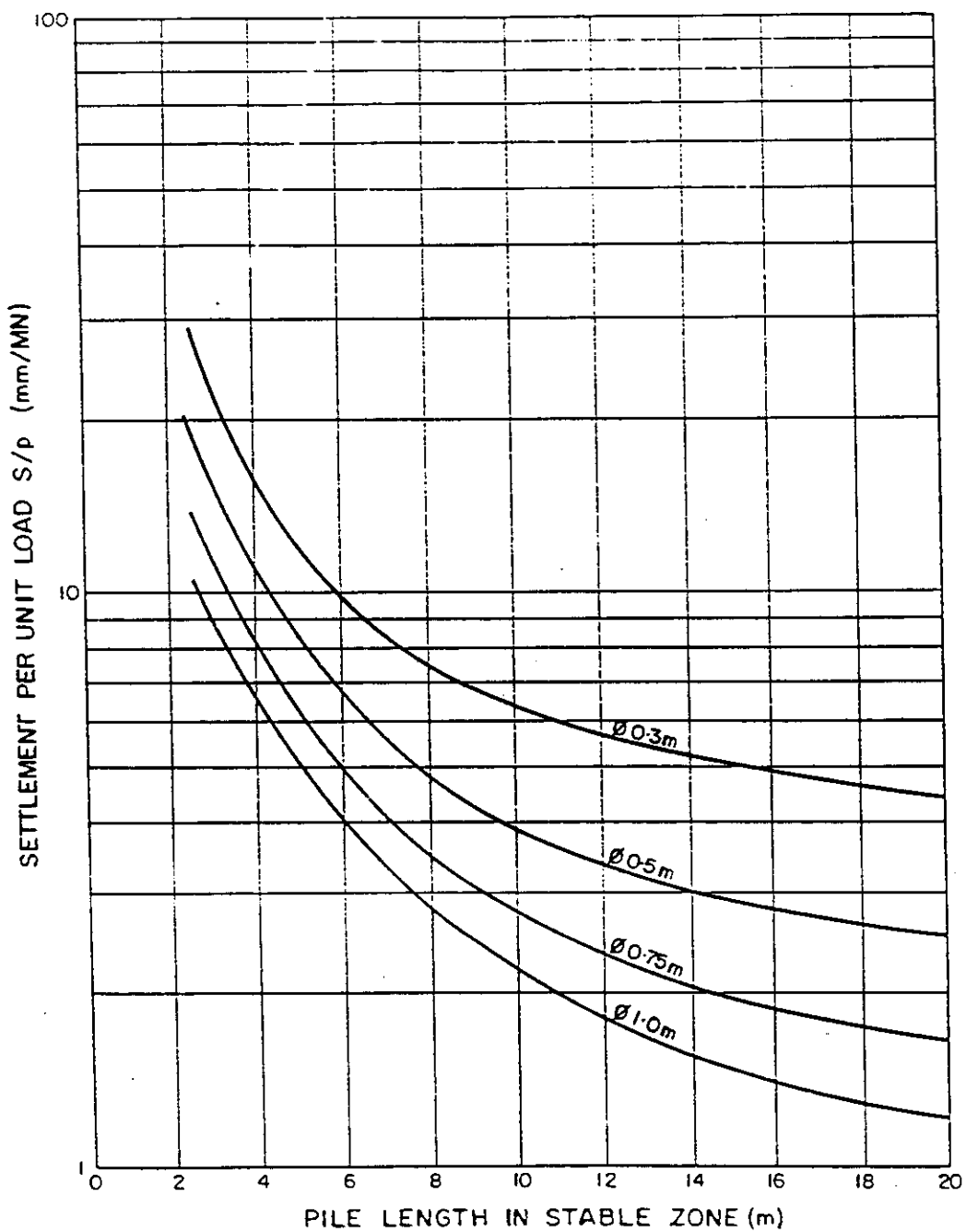
Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development

**Design Chart for Settlement of Single
 Driven Pile in Medium-Dense Sand**

GEOMARINE

**Figure
 5.5**

COFFEY



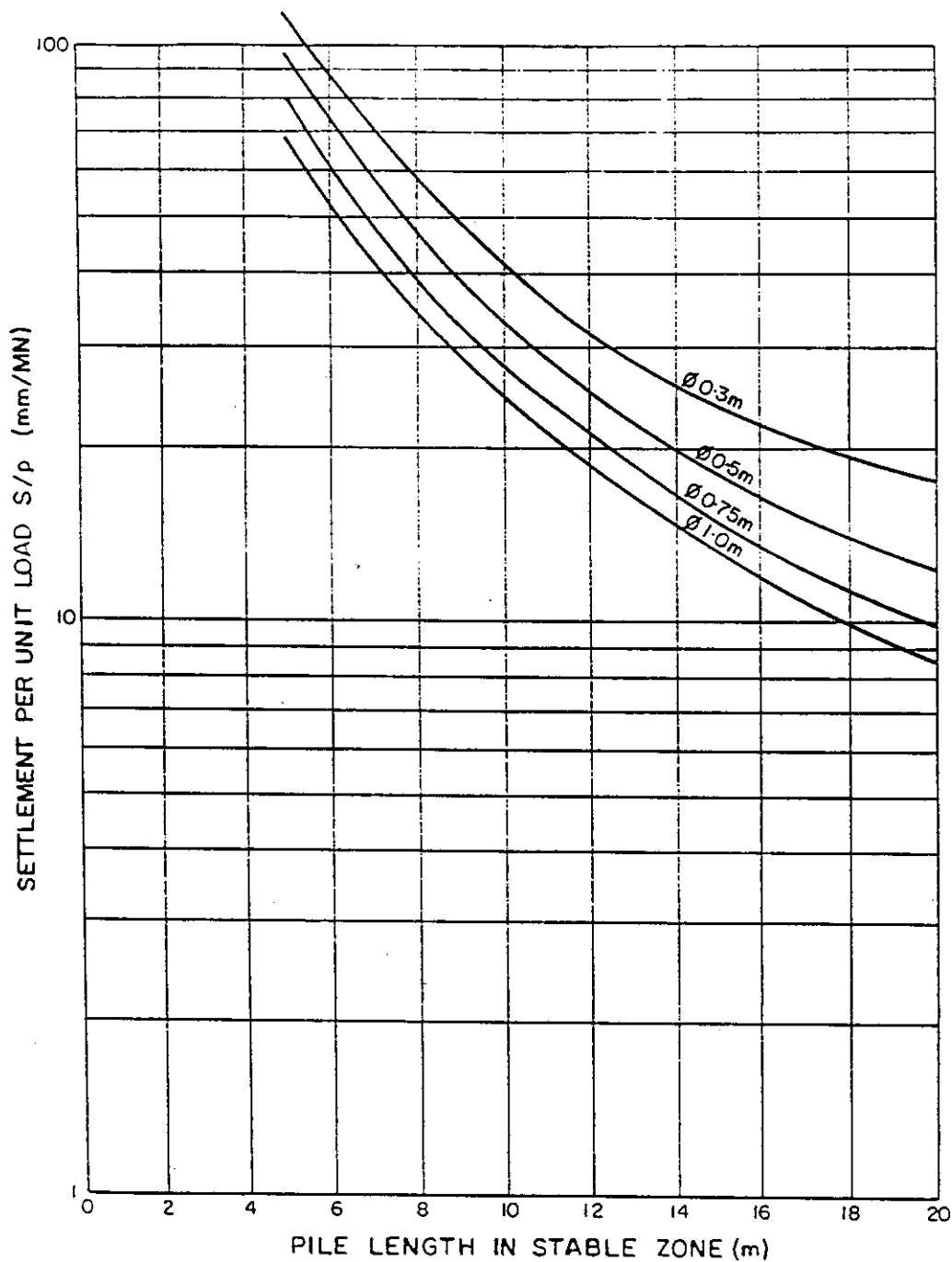
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Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Design Chart for Settlement of Single
 Driven Pile in Dense Sand

COFFEY

Figure
 5.6

GEOMARINE



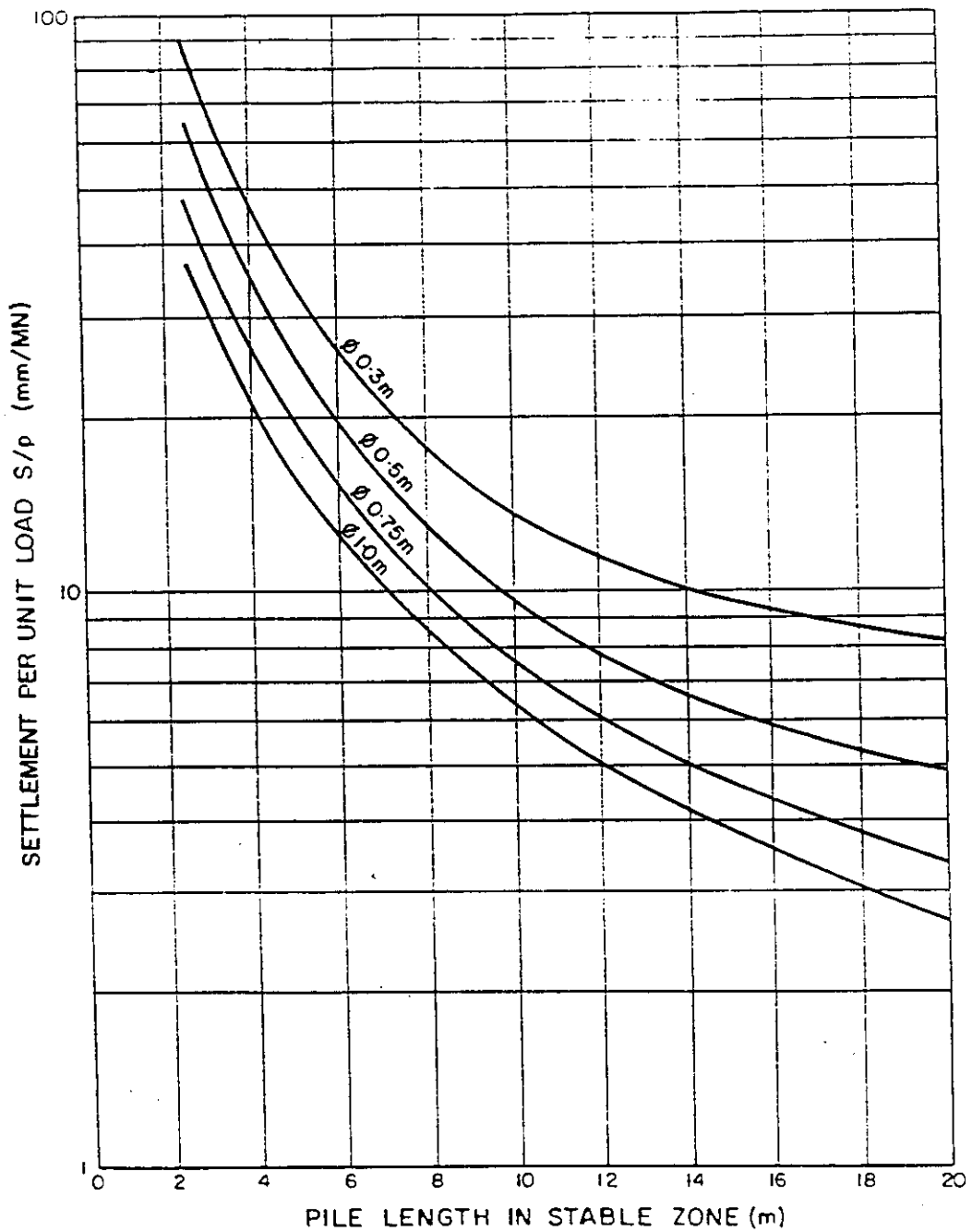
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Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
**Design Chart for Settlement of Single
 Bored Pile in Loose Sand**

GEOMARINE

**Figure
 5.7**

COFFEY



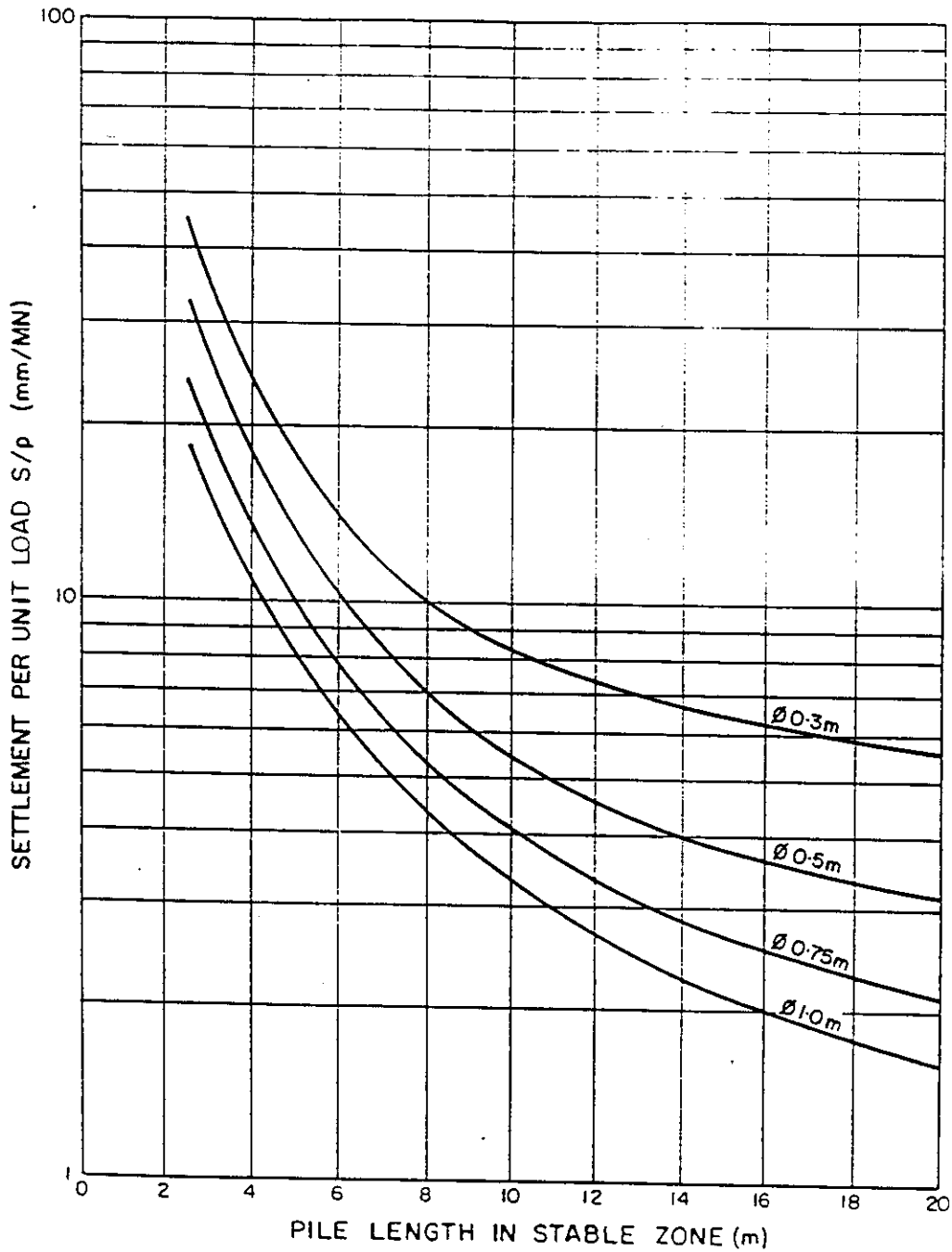
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Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Design Chart for Settlement of Single
 Bored Pile in Medium-Dense Sand

COFFEY

Figure
 5.8

GEOMARINE



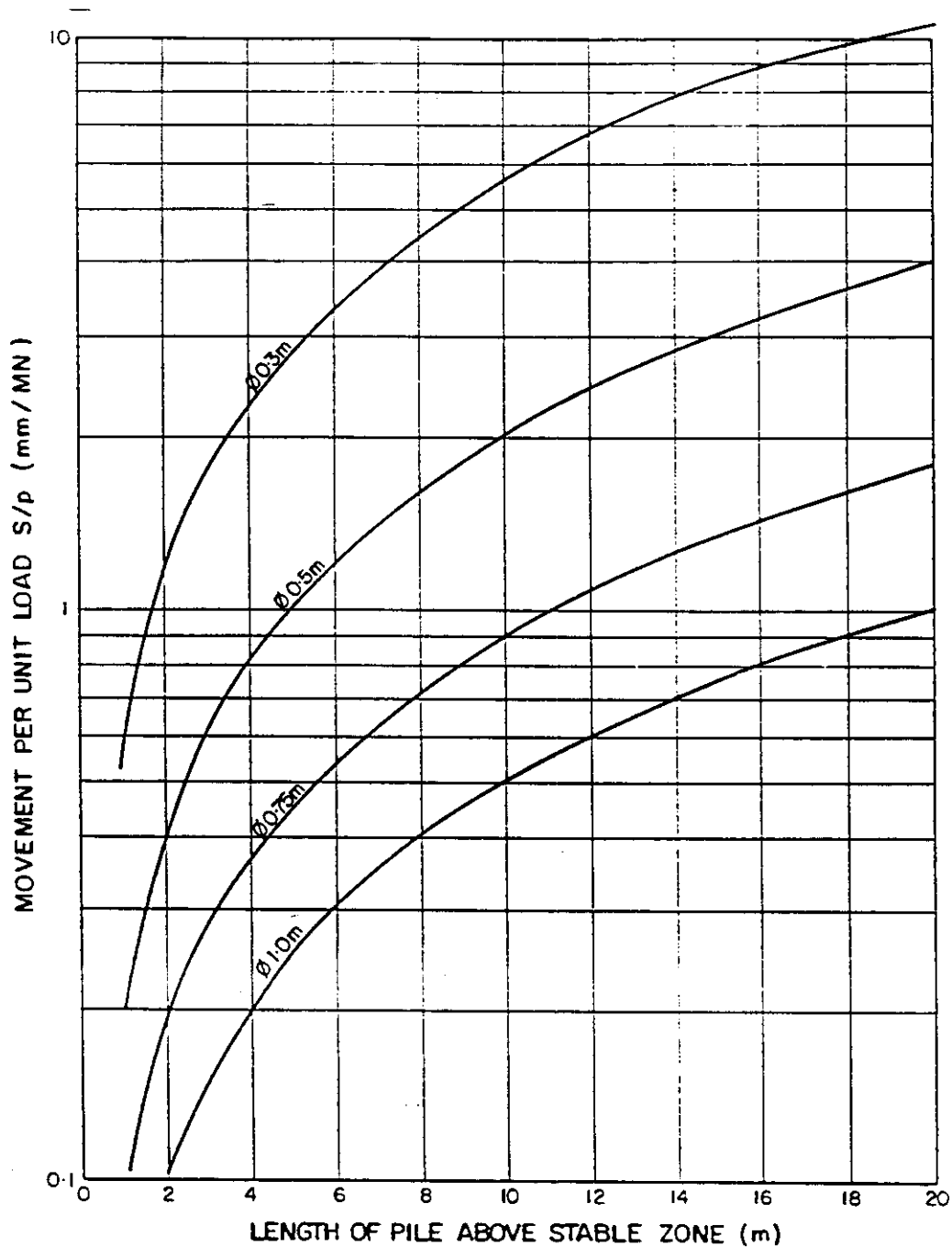
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Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
**Design Chart for Settlement of Single
 Bored Pile in Dense Sand**

GEOMARINE

Figure
5.9

COFFEY



NOTE - ASSUMED PILE MODULUS = 25 000 MPa

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Narrabeen-Collaroy-Fishermans Beach
 Foundation Design Criteria for Residential Development
 Design Chart for Pile Head Movement due to
 Compression above Stable Zone

COFFEY

Figure
 5.10

GEOMARINE

Appendix A

Derivation of Forces on Piling

Equations for Forces Acting on a Pile Due to a Collapsing Vertical Sand Face

It is assumed that the soil fails along a plane sliding surface (i.e. that "wedge" failure of the soil occurs), as shown in **Figure A1(a)**, and that the resulting pressure on the pile is as shown in **Figure A1(b)**. From statics, the total force acting on the upper part of the pile (to depth h) is (for a cohesionless soil):

$$F_1 = 0.5 B h^2 \gamma \cot\beta \tan(\beta - \phi) \quad \dots (A1)$$

where:

- B = width of failing wedge;
- γ = unit weight of soil;
- h = height of soil which collapses past the pile;
- β = slope angle of soil wedge; and
- ϕ = angle of internal friction of soil.

Similarly, the force F_2 on the lower part of the pile (from depth h to h_s) is:

$$F_2 = F_1 (h_s/h - 1) \quad \dots (A2)$$

where:

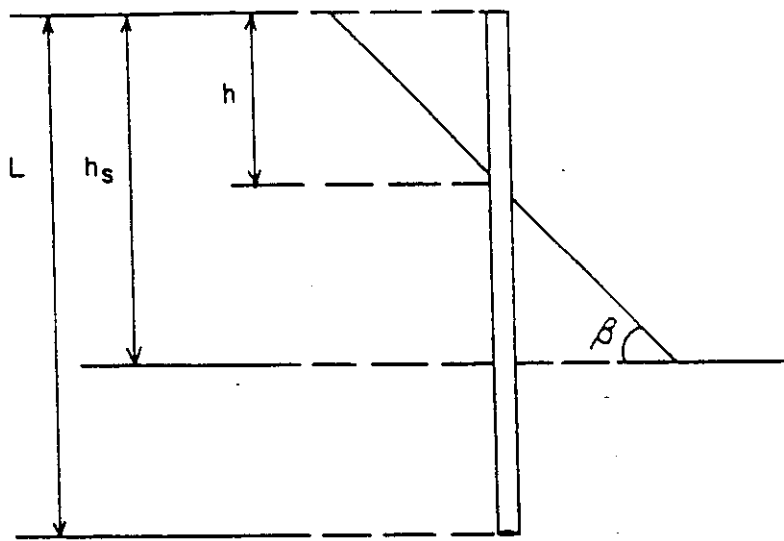
- h_s = depth of scour (or erosion) of soil.

It can be shown that, for the postulated mechanism, the maximum force on the pile occurs when $h = h_s/2$. For the particular case, the maximum shear force, V_{max} , and maximum bending moment, M_{max} , in the pile above the stable soil surface are expressed as follows:

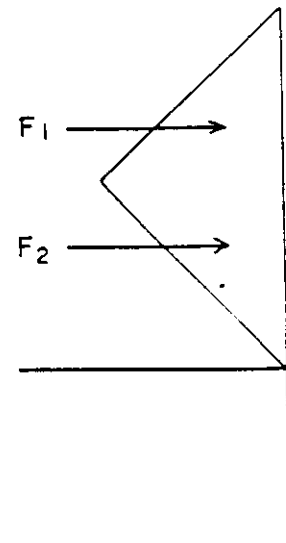
$$V_{max} = 0.25 h_s^2 B \gamma \cot\beta \tan(\beta - \phi) \quad \dots (A3)$$

$$M_{max} = 0.125 h_s^3 B \gamma \cot\beta \tan(\beta - \phi) \quad \dots (A4)$$

For calculations involving piles in moist beach sand it is considered that $\phi = 30^\circ$, $\beta = 45^\circ$ and $B = 3d$ (where d = pile diameter or width) are usually reasonable design parameters.



(a) ASSUMED SLOPE FAILURE MECHANISM



(b) ASSUMED LATERAL PRESSURE DIAGRAM

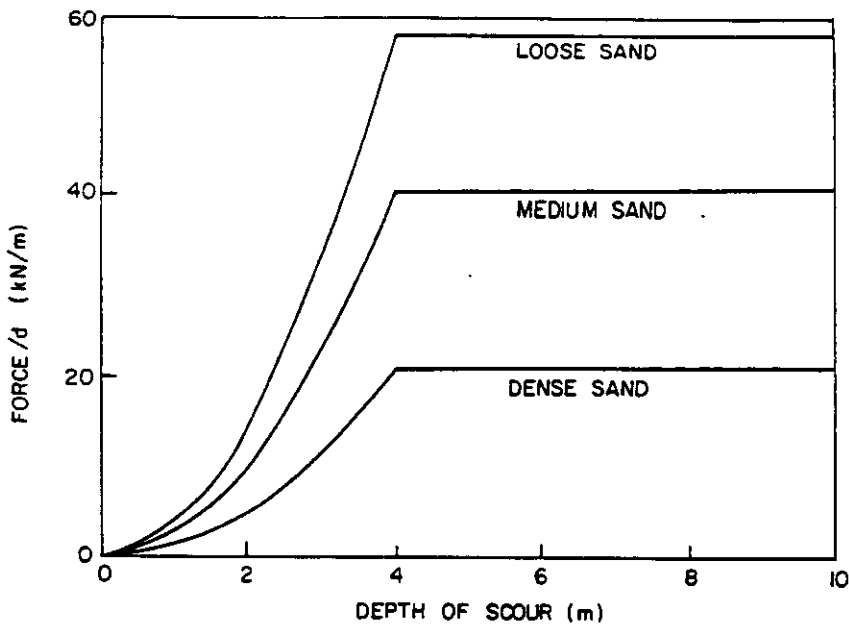
Coffey & Partners Pty Ltd Consulting Engineers in the geotechnical sciences

drawn	HGP/LT.
approved	
date	10/5/90
scale	

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 BEACH DEVELOPMENT DESIGN CRITERIA
 NARRABEEN TO COLLAROY BEACH
 ASSUMED FAILURE MECHANISM AND
 LATERAL PRESSURE DISTRIBUTION

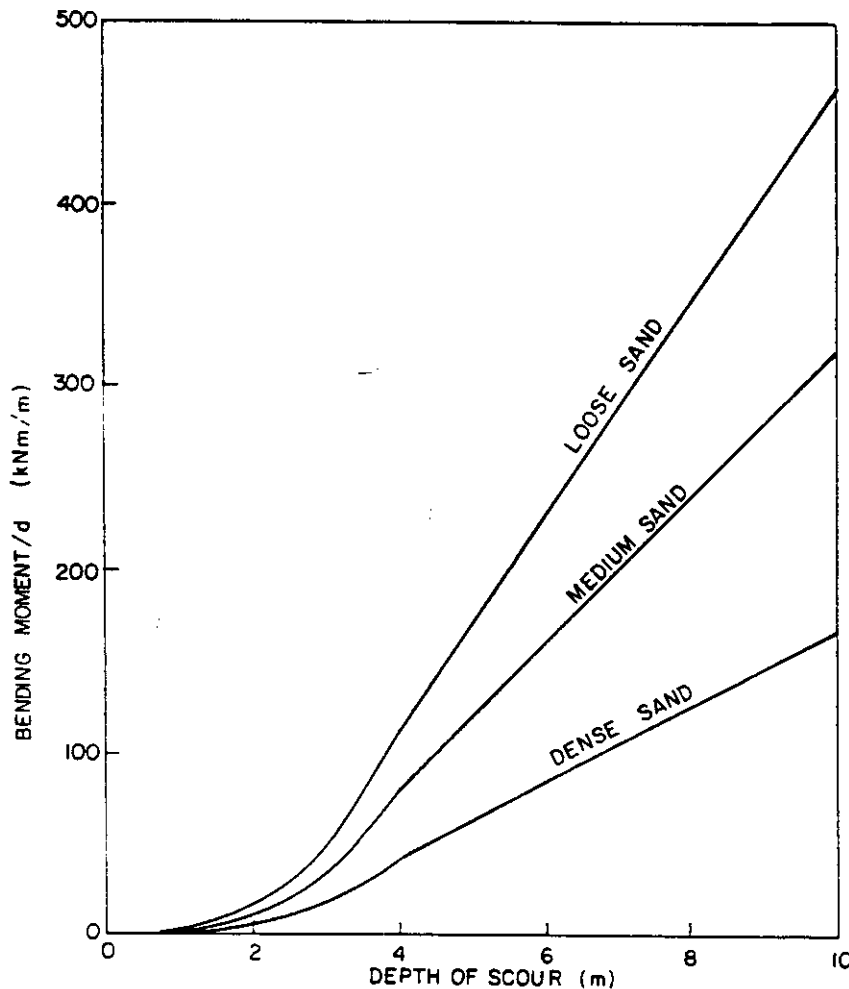


FIGURE A1
 job no: S7589/1



ASSUMED
PARAMETERS
 $\beta = 45^\circ$
 $B = 3d$

a) SHEAR FORCE



b) BENDING MOMENT AT SCOUR LEVEL

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Incorporated in NSW

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approved	<i>[Signature]</i>
date	10/1/91
scale	

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BEACH DEVELOPMENT DESIGN CRITERIA
NARRABEEN TO COLLAROY BEACH
EFFECT OF SAND DENSITY ON SHEAR FORCE
AND BENDING MOMENTS DEVELOPED IN
PILE DUE TO SLUMPING FACE



FIGURE A2

job no: S9425/1

Appendix B

Field Data

Geotechnical Investigations for the Narrabeen-Collaroy- Fishermans Beach Areas

A field investigation at specific sites was carried out between 30th November and 4th December 1990. Field work consisted of an auger drilling programme, together with electric friction cone testing. Eight holes were drilled to maximum depths of 10m at locations shown on **Figure 5.1**.

Drilling was carried out using an Edson 3000 truck mounted drilling rig equipped with continuous spiral flight augers. SPT tests were carried out in sands at intervals of 1.5m. The samples returned were retained for laboratory testing. Drilling, sampling and testing of the boreholes was carried out in the full time presence of an Engineering Geologist who produced engineering logs of all holes. These logs are presented together with explanation sheets defining the terms and symbols used in their preparation.

The holes were located by taking tape measurements relative to existing site features which are presented in "Borehole Location Sketches". Electric friction cone probes were carried out at each borehole location using a truck mounted "MACSIL" electric friction cone penetrometer. The results of these tests are presented. In general, the results of the cone penetration tests are consistent with the material characteristics inferred from the borehole logs and from the standard penetration test (SPT) data.



descriptive terms soil and rock

SOIL DESCRIPTIONS

Classification of Material based on Unified Classification System (refer SAA Site Investigation Code AS1726-1975 Add. No. 1 Table D1).

Moisture Condition based on appearance of soil

- dry** Looks and feels dry; cohesive soils usually hard, powdery or friable, granular soils run freely through hands.
- moist** Soil feels cool, darkened in colour; cohesive soils usually weakened by moisture, granular soils tend to cohere, but one gets no free water on hands on remoulding.
- wet** Soil feels cool, darkened in colour; cohesive soils weakened, granular soils tend to cohere, free water collects on hands when remoulding.

Consistency based on unconfined compressive strength (Qu) (generally estimated or measured by hand penetrometer)

term	very soft	soft	firm	stiff	very stiff	hard
Qu kPa	25	50	100	200	400	

If soil crumbles on test without meaningful result, it is described as friable.

Density Index

(generally estimated or based on penetrometer results).

term	very loose	loose	medium dense	dense	very dense
density index I _D %	15	35	65	85	

ROCK DESCRIPTIONS

Weathering based on visual assessment

term	criterion
Fresh:	Rock substance unaffected by weathering.
Slightly Weathered:	Rock substance affected by weathering to the extent that partial staining or partial discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable; strength properties are essentially those of the fresh rock substance.
Moderately Weathered:	Rock substance affected by weathering to the extent that staining extends throughout whole of the rock substance and the original colour of the fresh rock is no longer recognisable.
Highly Weathered:	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and signs of chemical or physical decomposition of individual minerals are usually evident. Porosity and strength may be increased or decreased when compared to the fresh rock substance, usually as a result of the leaching or deposition of iron. The colour and strength of the original fresh rock substance is no longer recognisable.
Extremely Weathered:	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.

Strength based on point load strength index, corrected to 50 mm diameter - I_s(50) (refer I.S.R.M., Commission on Standardisation of Laboratory and Field Tests, Suggested Methods for Determining the Uniaxial Compressive Strength of Rock Materials and the Point Load Strength Index, Committee on Laboratory Tests Document No. 1). (Generally estimated: x indicates test result).

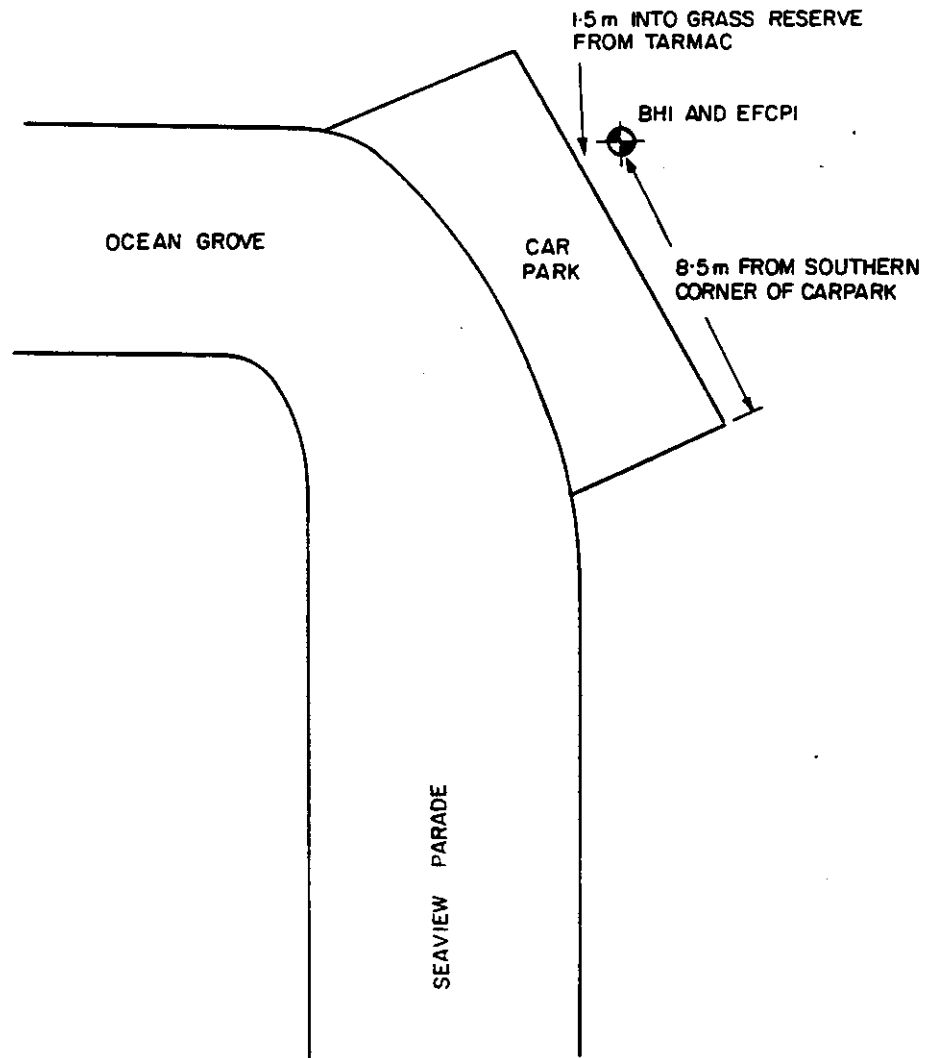
classification	extremely low	very low	low	medium	high	very high	extremely high
I _s (50) MPa	0.03	0.1	0.3	1	3	10	

The unconfined compressive strength is typically about 20 x I_s(50) but the multiplier may range, for different rock types, from as low as 4 to as high as 30.

Defect Spacing

classification	extremely close	very close	close	medium	wide	very wide	extremely wide
spacing m	0.03	0.1	0.3	1	3	10	

Defect description uses terms contained on AS1726 table D2 to describe nature of defect (fault, joint, crushed zone, clay seam etc.) and character (roughness, extent, coating etc.).



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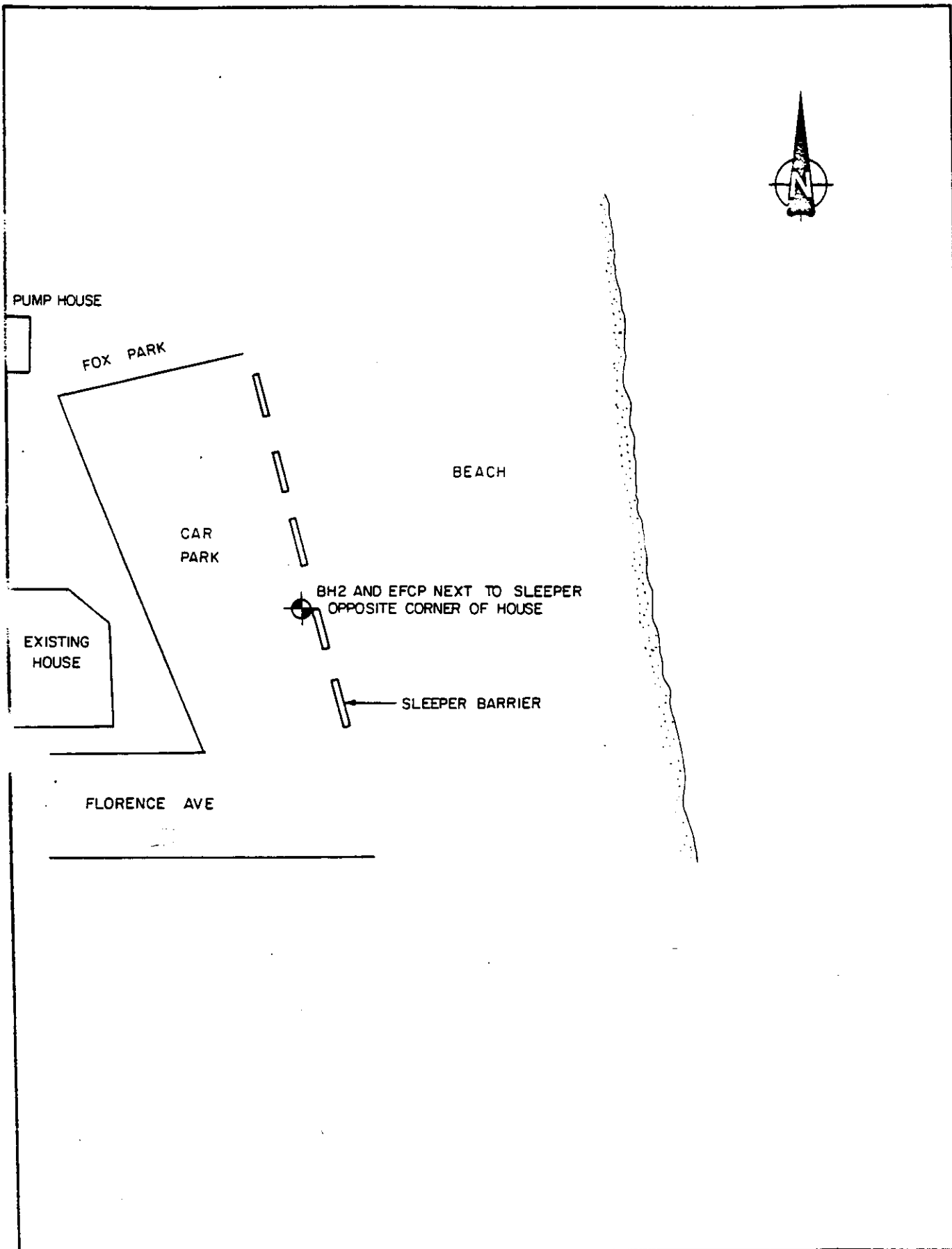
drawn	SRM / LT
approved	
date	10/1/91
scale	NTS

GEOMARINE PTY LTD
BEACH DEVELOPMENT DESIGN CRITERIA
NARRABEEN TO COLLARROY BEACH
LOCATION OF BOREHOLE 1



FIGURE B1

job no: S9425/1



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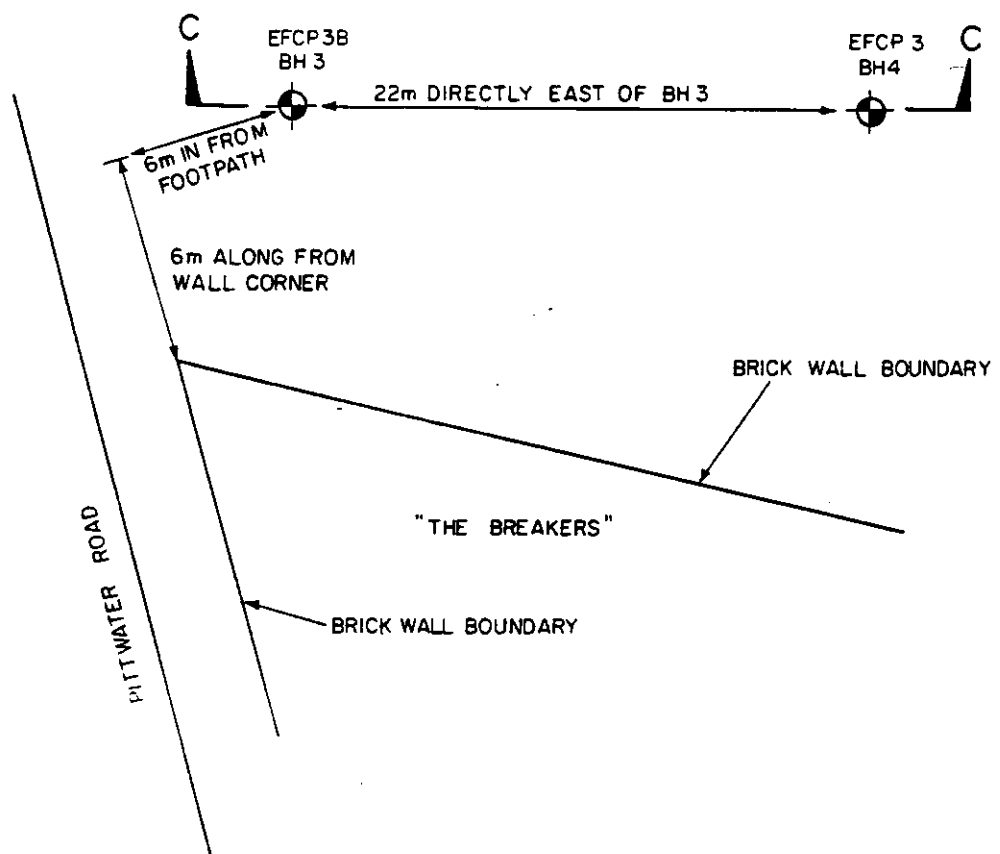
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approved	<i>[Signature]</i>
date	10/1/91
scale	NTS

GEOMARINE PTY LTD
BEACH DEVELOPMENT DESIGN CRITERIA
NARRABEEN TO COLLAROY BEACH
LOCATION OF BOREHOLE 2



FIGURE B2

job no: S94 25/1



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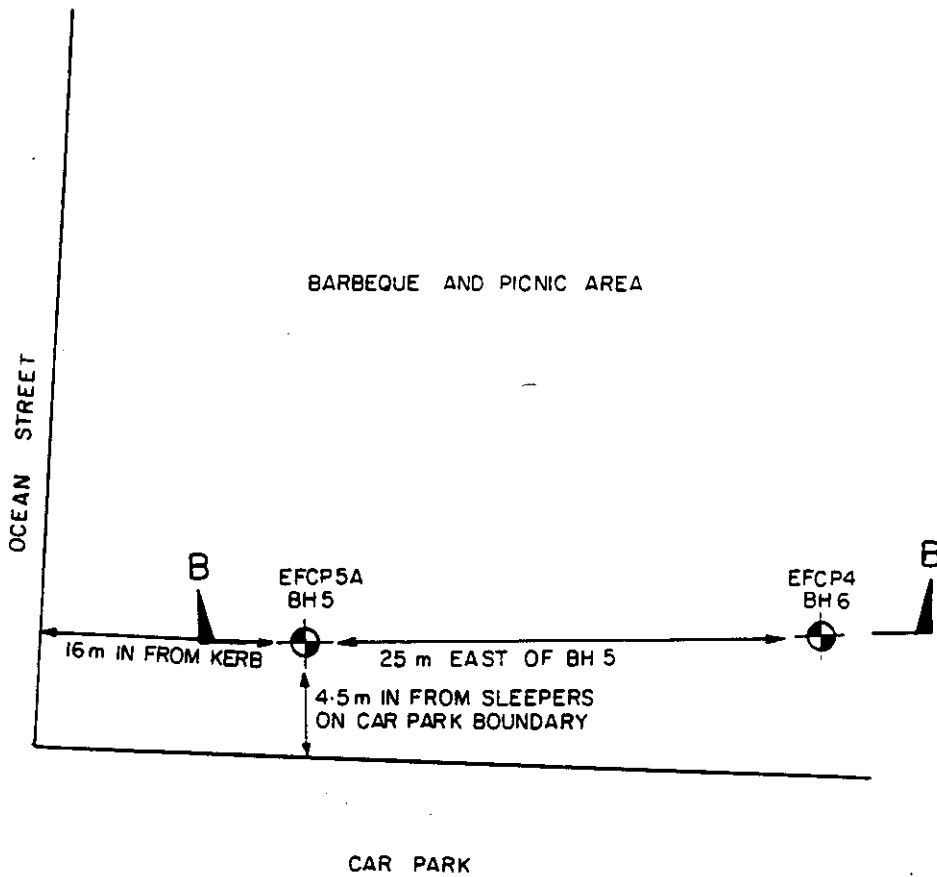
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drawn	SRM / LT
approved	
date	10/1/91
scale	NTS

GEOMARINE PTY LTD
BEACH DEVELOPMENT DESIGN CRITERIA
NARRABEEN TO COLLAROY BEACH
LOCATION OF BOREHOLES 3 AND 4



FIGURE B3
job no: S9425/1



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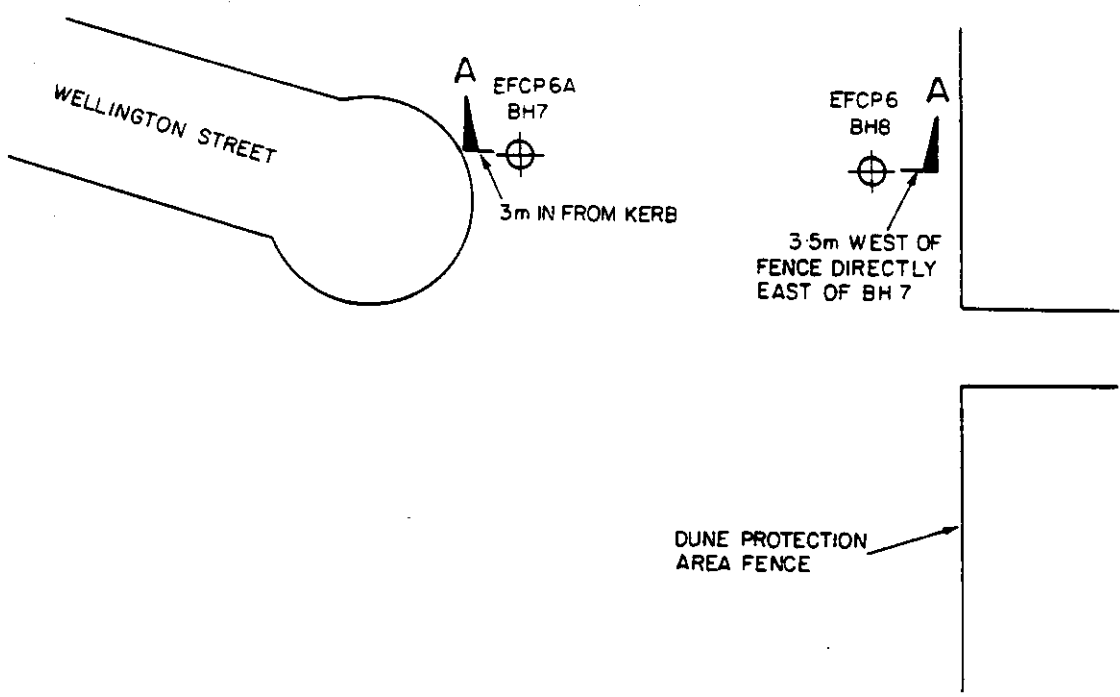
drawn	SRM /LT
approved	
date	10/1/91
scale	NTS

GEOMARINE PTY LTD
BEACH DEVELOPMENT DESIGN CRITERIA
NARRABEEN TO COLLAROY BEACH
LOCATION OF BOREHOLES 5 AND 6



FIGURE B4

job no: S9425/1



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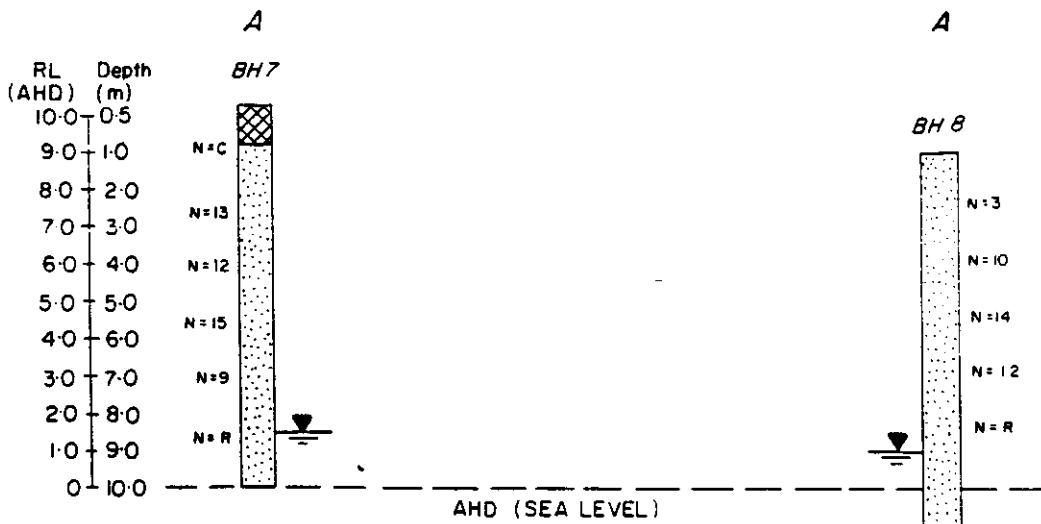
drawn	SRM / LT
approved	<i>[Signature]</i>
date	10/1/91
scale	NTS

GEOMARINE PTY LTD
BEACH DEVELOPMENT DESIGN CRITERIA
NARRABEEN TO COLLAROY BEACH
LOCATION OF BOREHOLES 7 AND 8

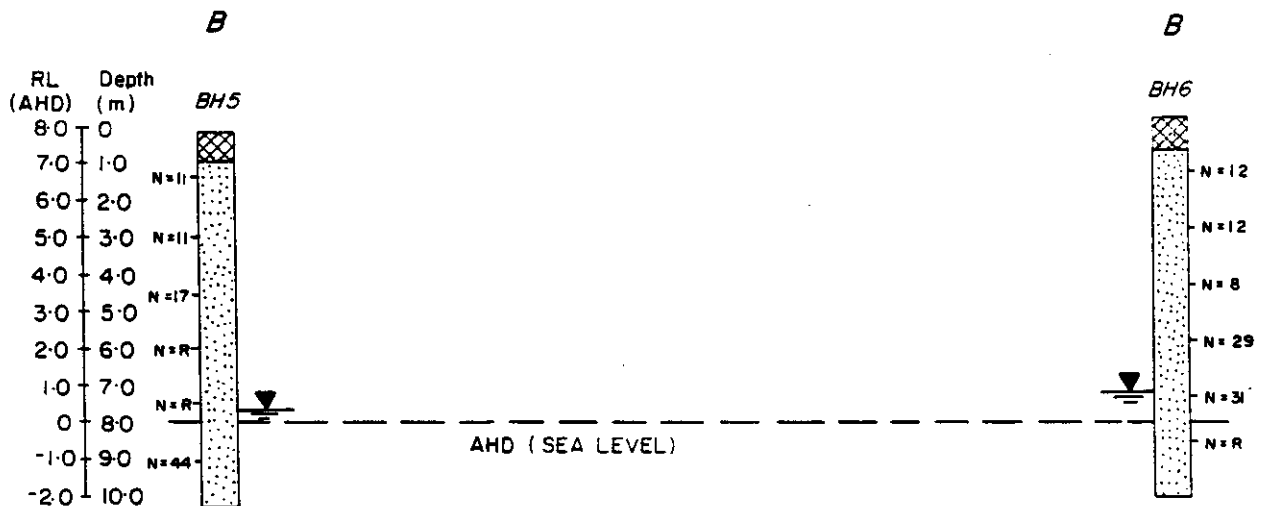


FIGURE B5

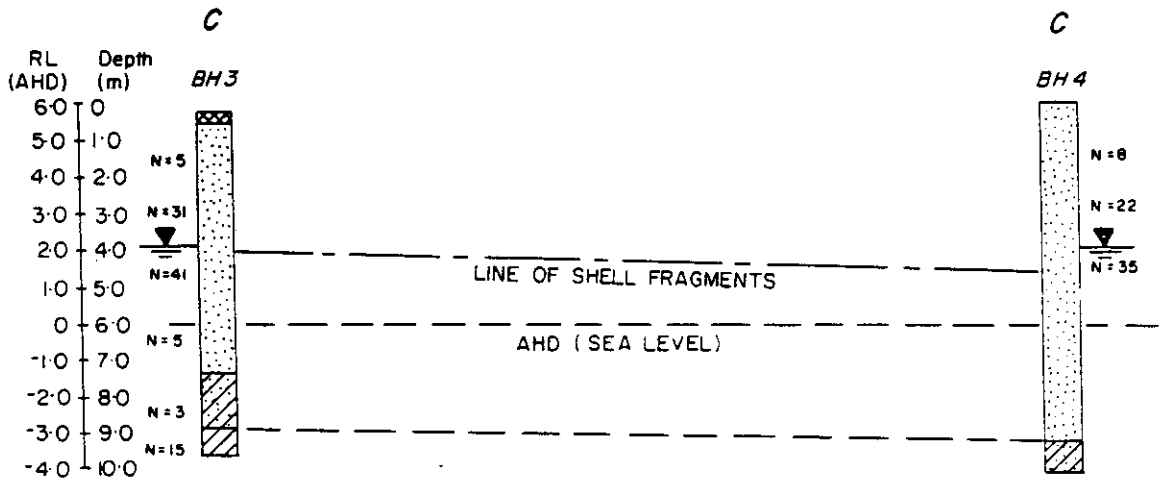
job no: S9425/1



CROSS SECTION A - A



CROSS SECTION B - B



CROSS SECTION C - C

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drawn	SRM/LT
approved	<i>[Signature]</i>
date	10/1/91
scale	

GEOMARINE PTY LTD
BEACH DEVELOPMENT DESIGN CRITERIA
NARRABEEN TO COLLAROY BEACH
CROSS SECTIONS AA, BB AND CC



FIGURE B6

job no: S9425 / 1



borehole no
BH1
sheet 1 of 2

engineering log - borehole

office job no: S9421
date commenced: 4/12/99
date completed: 4/12/99
logged by: SRM
checked by: MAE

client: GEOMAPINE PTY LTD
practice: WAPPINGAH SHIRE COUNCIL
project: BEACH DEVELOPMENT DESIGN CRITERIA
borehole location: SEE DRAWING No. 81

drill model and mounting: EDSON 3000 - BRUCH slope: -90 DEG R.L. Surface: 6.3 m
hole diameter: 100mm bearing: datum: AHD

method	penetration	support	notes samples tests etc	PL	depth meter	penetration meter	classification symbol	material soil type/plasticity or particle characteristics colour, secondary and minor components	moisture condition	consistency/density index	structure and additional observations
AD1					0			FILL Sand fine to medium grained, grey brown, some gravel	M		FILL
					1		SM	TOPSOIL Silty Sand fine to medium grained, dark grey		MD	TOPSOIL
			4 3 2 N ^o = 5		7		SC	CLAYEY SAND fine to medium grained, light grey - yellow Clay high plasticity			ALLUVIUM
					2						
			5 4 2 N ^o = 1		6		CH	CLAY high plasticity, grey & red brown, trace amounts of sand	>Wp		ALLUVIUM
					3						
			5 6 7 N ^o = 13		4					H	
					4						
			5 6 7 N ^o = 13		5		CL	CLAY medium plasticity, grey red yellow mottled, trace of fine sand	<W _L		RESIDUAL CLAY with relic structures Blocky texture
					6						
			5 6 7 N ^o = 10		7					H	
					8			becoming EW claystone, red brown			
					9						
			5 6 7 N ^o = 10		10						

METHOD	SUPPORT	NOTES	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION	CONSISTENCY/DENSITY INDEX
AS auger screw log	...	US undisturbed sample 50 mm diameter		VS very soft
AD auger drilling	...	D disturbed sample		S soft
P roller method		R standard penetration test		F firm
W washbore		N ^o SPT - sample recovered		St stiff
CT cable tool		N ^o SPT with solid cone		VS very stiff
HA hand auger		V vane shear		H hard
DT diatube		P pressuremeter		Fa friable
T bit shown by suffix		Bs bulk sample		VL very loose
e diat bit		R refusal		L loose
v v bit				MD medium dense
T TC bit				D dense
e.g. AD1				VD very dense

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borehole no
BH1
sheet 2 of 2

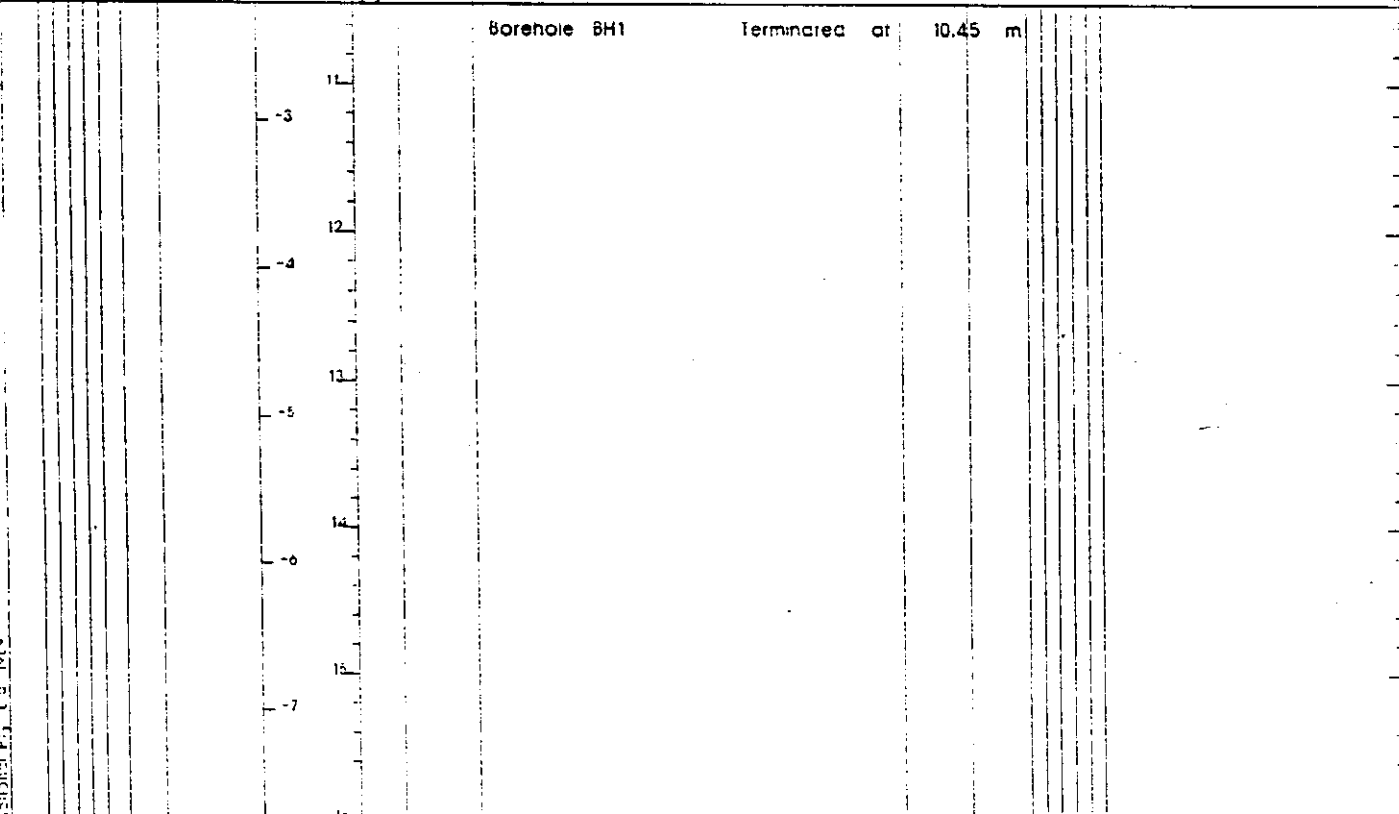
engineering log - borehole

OFFICE JOB NO 89423 1

Client	GEOMARINE PTY LTD	hole commenced	4 12 98
Principal	WARRINGAH SHIRE COUNCIL	hole completed	4 12 98
Project	BEACH DEVELOPMENT DESIGN CRITERIA	logged by	SRM
borehole location	SEE DRAWING NO B1	checked by	MAD

drill model and mounting	EDSON 3000 TRUCK	drill pipe	99 DEG	R/L Surface	53 m
hole diameter	100mm	drill bit		color	AHD

depth (m)	depth (ft)	notes (samples, test etc.)	R/L	d (mm)	penetration (mm)	penetration (ft)	resistance (kN)	resistance (lb)	material	moisture (%)	density (Mg/m ³)	bound water (%)	structure and additional observations
0	0								CLAY medium plastic, yellow mottled trace of fine sand	M	H		RESIDUAL CLAY with relic structures
0.6	0.6	N=32							some bands of ironstone				
1.3	1.3	N=15											
10.45	10.45												SPT refusal: 22 blows for 50mm



METHOD AS auger screwing* AD auger drilling* R roller-tricone W washbore CI cable tool HA hand auger DI diatube *bit shown by suffix B blank bit V V bit T TC bit e.g. ADI	SUPPORT C casing M mud PENETRATION no resistance ranging 1: refusal WATER not measured water level water outflow water inflow	NOTES samples and tests U50 undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N* SPT - sample recovered Nc SPT with solid cone V vane shear P pressuremeter Bs bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wc plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VS+ very stiff H hard Fp friable Vl very loose L loose MD medium dense D dense VD very dense
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borehole no
BH2
sheet 1 of 1

engineering log - borehole

office job no: S9425.1

client: GEOMARINE PTY LTD	hole commenced: 30/11/90
principal: WARRINGAH SHIRE COUNCIL	hole completed: 30/11/90
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SRM
borehole location: SEE DRAWING No B2	checked by: MAB

drill model and mounting: EDSON 3000 - TRUCK	slope: -90 DEG	R.L. Surface: 4.1 m
hole diameter: 100mm	bearing:	datum: AHD

method	penetration	support	notes samples test, etc	depth metres	graphic log	classification symbol	material soil type, plasticity or particle characteristics colour, secondary and minor components	moisture condition	consistency/density index	hand penetrometer	structure and additional observations
ADT		NIL		4	[Cross-hatched pattern]		FILL: Sand medium to coarse grained red brown, with some gravel	M			FILL sandstone & concrete gravel
				3	[Cross-hatched pattern]		Layer of coarse gravel 100mm thick				Beach sand contains shell fragments
			U 1.1 N*=2	2	[Dotted pattern]	SP	SAND medium to coarse grained yellow brown, some fine gravel		L		Beach sand contains shell fragments
			U 1.0 N*=1	1	[Dotted pattern]						
				0	[Diagonal lines]	SP	CLAYEY SAND fine to coarse grained, yellow brown, trace of fines Gravelly layer 300mm thick				SPT travelled 450mm with one blow
				-1	[Horizontal lines]		CLAYSTONE yellow red grey very low strength EV - HW				EW - HW CLAYSTONE
				-2	[Horizontal lines]		Becoming carbonaceous, HW very low strength				
				-3	[Horizontal lines]		Borehole BH2 Terminated at 5.70 m				

METHOD AS auger screwing AD auger drilling R roller tricone W washpore CT cable tool HA hand auger DT airprobe *bit shown by suffix B blank bit V V bit T TC bit e.g. ADT	SUPPORT C casing M mud PENETRATION 1 2 3 no resistance ranging to refusal WATER • not measured water level water outflow water inflow	NOTES samples and tests USC unaltered sample 50 mm diameter D disturbed sample H standard penetration test N* SPT + sample recovered Nc SPT with solid cone V vane shear P pressuremeter Bs bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VS1 very stiff H hard Fb friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no
BH3
sheet 1 of 2

engineering log - borehole

office job no: S9425-1

client: GEOMARINE PTY LTD	hole commenced: 4.12.90
principal: WARRINGAH SHIRE COUNCIL	hole completed: 4.12.90
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SRM
borehole location: SEE DRAWING No B3	checked by: MAB

drill model and mounting: EDSON 3000 - TRUCK size: -90 DEG R.L. Surface: 5.8 m
 hole diameter: 100mm bearing: datum: AHD

method	penetration	support	water	notes samples, test, etc	RL	depth metres	graphic log	classification symbol	material soil type; plasticity or particle characteristics colour, secondary and minor components	moisture condition	consistency/density index	hand penetration meter	structure and additional observations
ADT	1 2 3	NIL				5	[Cross-hatched]	SP	FILL: Sand fine to medium grained, brown, trace of gravel	M			FILL
				2 . 2 . 3 N*=5		1	[Dotted]	SP	SAND: fine to medium grained, grey brown clayey layer at 1m		MD		DUNE SAND
						4	[Dotted]	SP	SAND: fine to medium grained, orange brown				
				8 . 15 . 10 N*=31		3	[Dotted]		Beach sand with calcareous cementation		VD		
						2	[Dotted]	SP	SAND: medium grained, yellow brown, shells & trace of gravel				Beach sand with calcareous cementation
				5 . 12 . 29 N*=41		4	[Dotted]			W			Hole collapsing below water table
						1	[Dotted]						
				3 . 2 . 3 N*=5		0	[Dotted]						SPT at 5.9m returned empty
						-1	[Dotted]						
						-2	[Diagonal lines]	SC	CLAYEY SAND: fine to medium grained, dark grey, clay high plasticity, trace of organics				Estuarine or marine deposits

METHOD AS auger screwing AD auger drilling R roller-tricone W washbore CT cable tool HA hand auger DT diatube *bit shown by suffix B plank bit V V bit T IC bit e.g. ADT	SUPPORT C casing M mud PENETRATION 1 2 3 no resistance hanging to refusal WATER * not measured water level water outflow water inflow	NOTES samples and tests US0 undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N* SPT + sample recovered Nc SPT with solid cone V vane shear P pressuremeter Bs bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VSt very stiff H hard Fb friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no
BH3
sheet: 2 of 2

engineering log - borehole

office job no: S6420 1

client: GEOMARINE PTY LTD	hole commenced: 4 12 90
principal: WARRINGAH SHIRE COUNCIL	hole completed: 4 12 90
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SRM
borehole location: SEE DRAWING NO 103	checked by: MAD

drill model and mounting: EDSON 3000 - TRUCO	slope: -90 DEG	R.L surface: 5.8 m
hole diameter: 100mm	bearing:	datum: AHD

method	penetration	SUPPORT	water	notes samples, test, etc	RL	depth metres	classification	material soil type; plasticity or particle characteristics colour, secondary, and minor components	moisture condition	consistency/density index	hand penetrometer	structure and additional observations
W							SC	CLAYEY SAND: fine to medium grained, dark grey clay high plasticity trace of organics	01			
							CH	CLAY high plasticity, grey & brown, with some sand fine to medium grained trace of organics		SI		ESTUARINE CLAY
								Borehole BH3 Terminated at	9.35	VS1		

METHOD AS auger screwing AD auger drilling R roller tinsone W washbore CT cable tool HA hand auger DI dialube *bit shown by suffix E blank bit V V bit TC TC bit e.g AD1	SUPPORT M mud PENETRATION R resistance ranging 10 R refusal WATER * not measured water level water outflow water inflow	NOTES samples and tests US undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N+ SPT + sample recovered NC SPT with sand cone V vane shear P pressuremeter B bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VS1 very stiff H hard Fb friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no:
BH4
sheet 1 of 2

engineering log - borehole

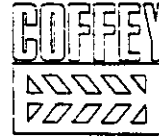
office job no S9425 1

client: GEOMARINE PTY LTD	hole commenced: 4 12 90		
principal: WARRINGAH SHIRE COUNCIL	hole completed: 4 12 90		
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SRM		
borehole location: SEE DRAWING No 02	checked by: MAB		
drill model and mounting: EDSON 3000 - TPUCK	slope: -90 DEG	R L Surface	0 1 m
hole diameter: 100mm	bearing:	datum:	AHD

method	penetration	support	water	notes samples, test, etc	RI	depth metres	graphic log	classification symbol	material: soil type, plasticity or particle characteristics, colour, secondary and minor components	moisture condition	consistency/density index	hand k. g. electro. meter	structure and additional observations
AD1		MIL			6			SP	SAND: fine to medium grained, light yellow brown, little or no fines	M			DUNE SAND
				4 4 4 N*=8	5	1					D		
				5 10 12 N*=22	3	3							
			▽	0 14 21 N*=35	2	4			Some coarser layers containing shells at 4.6m	W	VD		Hole collapsing before SPT at 4.6m
					1	5							
					U	6							
					-1	7			Trace amounts of gravel, sand coarser, some shell.				
								SP	SAND fine to medium grained, grey, some silt patches of clay.				ESTAUURINE CLAYEY SAND

METHOD AS auger screwing* AD auger drilling* P roller tricone W washbore CT cable tool HA hand auger DT diatube *bit shown by suffix B blank bit V V bit T TC bit e.g AD1	SUPPORT C casing M mud PENETRATION 1 2 3 no resistance ranging to refusal WATER not measured ▽ water level ▽ water outflow ▽ water inflow	NOTES samples and tests U50 undisturbed sample 50 mm diameter D disturbed sample N standard penetration test: N+ SPT + sample recovered Nc SPT with solid cone V vane shear P pressuremeter Bs bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system: MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VS1 very stiff H hard Fb friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no
BH4
sheet 2 of 2

engineering log - borehole

office job no. S9422.1

client: GEOMARINE PTY LTD	principal: WARRINGAH SHIRE COUNCIL	project: BEACH DEVELOPMENT DESIGN CRITERIA	borehole location: SEE DRAWING No B7	office job no. S9422.1	hole commenced: 4/12/90	hole completed: 4/12/90	logged by: SRM	checked by: MAE
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drill model and mounting: EDSON 300C - TRUCK	hole diameter: 100mm	slope: -90 DEG	R.L. Surface datum: 0.1 m
		bearing:	AHD

method	penetration	notes samples, test, etc	RI	depth metres	classification symbol	material soil type plasticity or particle characteristics colour, secondary and minor components	moisture condition	consistency/density index	hand penetrometer	structure and additional observations
ADT				0	SP	SAND fine to medium grained, grey some silt, patches of clay	W	VD		ESTAUURINE CLAYEY SAND
				10	CH	SANDY CLAY: medium to high plasticity grey, sand fine to medium grained	>Wp			ESTAUURINE CLAY
Borehole BH4 Terminated at 10.10 m										

METHOD AS auger screwing* AD auger drilling* P roller tricone W washbore CT cable tool HA hand auger DT diatube *bit shown by suffix D blank bit V V bit T TC bit e.g. ADT	SUPPORT L casing M mud PENETRATION WATER *not measured water level water outflow water inflow	NOTES samples and tests USL undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N+ SP1 + sample recovered No SP1 with solid cone V vane shear P pressuremeter Bt bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VS+ very stiff H hard Fc friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no.
BH5
sheet 1 of 2

engineering log - borehole

office job no S542: 1

client:	GEOMARINE PTY LTD	hole commenced:	4 12 90
principal:	WARRINGAH SHIRE COUNCIL	hole completed:	4 12 90
project:	BEACH DEVELOPMENT DESIGN CRITERIA	logged by:	SRM
borehole location:	SEE DRAWING B4	checked by:	MAB

drill model and mounting:	EDSON 3000 - TRUCK	slope:	-90 DEG	RL Surface:	7.5 m
hole diameter:	100mm	sealing:		datum:	AHD

method	penetration	support	notes samples, test, etc	R.L.	depth metres	graphic log	classification symbol	material soil type; plasticity or particle characteristics colour, secondary and minor components	moisture condition	consistency/density index		structure and additional observations
										penetration	penetration	
AD1	1	C			7	[Cross-hatched pattern]	SP	FILL: Sand fine to medium grained, grey brown, some gravel	M			FILL
	2		4, 5, 6 N*=11		1	[Dotted pattern]		SAND: fine to medium grained, yellow brown, little or no fines		D		DUNE SAND
	3		5, 5, 6 N*=11		2	[Dotted pattern]		Trace amounts of shell fragments below 3.2m		MD		
	4		6, 8, 9 N*=17		3	[Dotted pattern]		Some coarser bands		D		
	5		14, 24 N*=R		4	[Dotted pattern]		Some thin layers of organic material from 0.2m		VD		refusal of SPT at 5.9m in indurated sand
	6		10, 25 N*=R		5	[Dotted pattern]		Trace amount of sand below water table		W		
	7				6	[Dotted pattern]						
	8				7	[Dotted pattern]						
	9				8	[Dotted pattern]						
	10				9	[Dotted pattern]						
	11				10	[Dotted pattern]						
	12				11	[Dotted pattern]						
	13				12	[Dotted pattern]						
	14				13	[Dotted pattern]						
	15				14	[Dotted pattern]						
	16				15	[Dotted pattern]						
	17				16	[Dotted pattern]						
	18				17	[Dotted pattern]						
	19				18	[Dotted pattern]						
	20				19	[Dotted pattern]						
	21				20	[Dotted pattern]						
	22				21	[Dotted pattern]						
	23				22	[Dotted pattern]						
	24				23	[Dotted pattern]						
	25				24	[Dotted pattern]						
	26				25	[Dotted pattern]						
	27				26	[Dotted pattern]						
	28				27	[Dotted pattern]						
	29				28	[Dotted pattern]						
	30				29	[Dotted pattern]						
	31				30	[Dotted pattern]						
	32				31	[Dotted pattern]						
	33				32	[Dotted pattern]						
	34				33	[Dotted pattern]						
	35				34	[Dotted pattern]						
	36				35	[Dotted pattern]						
	37				36	[Dotted pattern]						
	38				37	[Dotted pattern]						
	39				38	[Dotted pattern]						
	40				39	[Dotted pattern]						
	41				40	[Dotted pattern]						
	42				41	[Dotted pattern]						
	43				42	[Dotted pattern]						
	44				43	[Dotted pattern]						
	45				44	[Dotted pattern]						
	46				45	[Dotted pattern]						
	47				46	[Dotted pattern]						
	48				47	[Dotted pattern]						
	49				48	[Dotted pattern]						
	50				49	[Dotted pattern]						

METHOD AS auger screwing* AD auger drilling* R roller/tricone W washbore CT cable tool HA hand auger DT diatube *bit shown by suffix B blank bit V V bit T TC bit e.g. AD1	SUPPORT C casing M mud PENETRATION 1 no resistance ranging to refusal 2 3 WATER * not measured ∇ water level ∇ water outflow ∇ water inflow	NOTES samples and tests U50 undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N* SPT - sample recovered Nc SPT with sand cone V vane shear P pressuremeter Bs bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VS: very stiff H hard Fb friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no
BH5
sheet 2 of 2

engineering log - borehole

office job no: S9425 1

client: GEOMARINE PTY LTD	note commenced: 4.12.90
principal: WARRINGAH SHIRE COUNCIL	note completed: 4.12.90
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SRM
borehole location: SEE DRAWING B4	checked by: MAB

drill model and mounting: EDSON 3000 - TRUCK	slope: -90 DEG	R.L Surface: 7.8 m
hole diameter: 100mm	bearing:	datum: AND

method	penetration	surface	water	notes samples test etc	depth metres	graphic log	classification symbol	material soil type plasticity or particle characteristics colour secondary and minor components	moisture condition	consistency/density index	hand penetrometer	structure and additional observations
							SP	SAND fine to medium grained yellow brown little or no fines	M	VD		DUNE SAND
								Slightly cemented sand with trace amounts of clay and shell fragments				
								borehole BH5 Terminated at 10.00 m				

METHOD AS under screwing AD auger, manual P roller-trombol W washbore CT cable tool HA hand auger DT drilprobe bit shown by suffix B blank pit Y Y bit T TC bit e.g. ADT	SUPPORT C casing M mud PENETRATION 1 2 3 100 resistance 1000 resistance refusal WATER not measured water level water outflow water inflow	NOTES samples and tests U0 undisturbed sample 50 mm diameter D disturbed sample R standard penetration test SP1 sample recovered SP1 with solid cone V vane shear P pressuremeter B bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE U dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm SI stiff VS+ very stiff H hard Fb friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no. **BH6**
sheet 1 of 2

engineering log - borehole

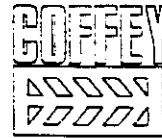
office job no. S9425 I

client: GEOMARINE PTY LTD	note commenced: 30/11/90
principal: WARRINGAH SHIRE COUNCIL	note completed: 30/11/90
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SRM
borehole location: SEE DRAWING B4	checked by: MAB
drill model and mounting: EDSON 3000 - TRUCK	slope: -90 DEG R.L. Surface: 8.2 m
bore diameter: 100mm	bearing: datum: AHD

method	penetration	support	water	notes samples, test, etc.	RL	depth metres	graphic log	classification symbol	material: soil type, plasticity or particle characteristics colour, secondary and minor components	moisture condition	consistency/density index	hand penetrometer	structure and additional observations
AD1		M			8				FILL: Sand fine to medium grained, light grey brown, some organics	D			FILL
	4, 5, 7 N*=12				7			SP	SAND: fine to medium grained yellow brown, little or no fines	M	D		DUNE SAND
	5, 6, 6 N*=12				6								
	5, 4, 4 N*=8				4						MD		
	6, 10, 19 N*=29				3			SP	SAND: fine to medium grained, brown, trace of organics in layers		D		DUNE SAND with some shells
	6, 15, 16 N*=31				1			SP	SAND: coarse grained, yellow brown with increasing shell content				INDURATED SAND

METHOD AS auger screwing* AD auger drilling* R roller, tricone W washbore CT cable tool HA hand auger DT dialube *bit shown by suffix B blank bit V V bit T TC bit e.g. AD1	SUPPORT C casing M mud PENETRATION no resistance remaining to refusal WATER * not measured ▽ water level ▽ water outflow ▽ water inflow	NOTES samples and tests U50 undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N* SPT + sample recovered Nc SPT with solid cone V vane shear P pressuremeter Bc bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VS1 very stiff H hard Fc friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no.
BH6
sheet 2 of 2

engineering log - borehole

office job no. S9425 1

client:	GEOMARINE PTY LTD	hole commenced:	30/11/90
principal:	WARRINGAH SHIRE COUNCIL	hole completed:	30/11/90
project:	BEACH DEVELOPMENT: DESIGN CRITERIA	logged by:	SRM
borehole location:	SEE DRAWING B4	checked by:	MAB

drill model and mounting:	EDSON 3000 - TRUCT	slide:	-90 DEG	R.L. Surface:	8.2 m
hole diameter:	100mm	bearing:		datum:	AMD

method	notes samples, test etc	RL	depth metres	graphic log	classification symbol	material soil type: plasticity or particle characteristics colour: secondary and minor components	moisture condition	consistency/density index	hand penetrometer	structure and additional observations
ADT	N* = R	0	0	[Symbol]	SP	SAND coarse grained, yellow brown, with increasing shell content	D	VD		INDURATED SAND
		-1	0			Increasing fines content, but mostly coarse sand				
		-10	10			Chunks of indurated sand returned on auger flights				
Borehole BH6 Terminated at 10.25 m										
		-11	11							
		-12	12							
		-13	13							
		-14	14							
		-15	15							
		-16	16							
		-17	17							

METHOD AS auger screwing* AD auger drilling* R roller-tricone W washpore CT cable tool HA hand auger DT airtube *bit shown by suffix B blank bit V V bit T TC bit eg ADT	SUPPORT C casing M mud PENETRATION 1 2 3 no resistance ranging to refusal WATER • not measured water level water outflow water inflow	NOTES samples and tests USC undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N* SPT + sample recovered Nc SPT with sand cone V vane shear P pressuremeter Bs bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system: MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VSt very stiff H hard Fc friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no:
BH7
sheet 1 of 2

engineering log - borehole

office job no: S6425 1

client: GEOMARINE PTY LTD	date commenced: 30 11 90
principal: WARRINGAH SHIRE COUNCIL	date completed: 30 11 90
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SPM
borehole location: SEE DRAWING B6	checked by: MAB

drill model and mounting: EDSON 3000 - TRUCK	slope: -0 DEG	R L Surface: 103 m
hole diameter: 105mm	bearing:	datum: AHD

method 1 2 3	penetration	support	water	notes samples, test, etc	RL	depth metres	graphic log	classification symbol	material: soil type; plasticity or particle characteristics colour, secondary and minor components	moisture condition	consistency density index	structure and additional observations
ADT						10		SP	FILL Sand fine to medium grained with some gravel	D		FILL
				2 2 2 N=6		9		SP	SAND, medium grained, light yellow brown, little or no fines	M	MD	DUNE SAND trace of shells
				4 6 7 N=13		8						
				4 5 7 N=12		7						
				7 9 6 N=15		6						
				4 4 2 N=9		5						
						4						
						3						

METHOD AS auger screwing AD auger drilling R roller tricone W washbore CT cable tool HA hand auger DT dialtube *bit shown by suffix B blank bit V V bit T TC bit eg ADT	SUPPORT C casing M mud PENETRATION 1 no resistance 2 gaining TC 3 refusal WATER * not measured water level water outflow water inflow	NOTES samples and tests USU undisturbed sample 50 mm diameter L disturbed sample N standard penetration test N* SPT + sample recovered NC SPT with solid cone V vane shear P pressuremeter BS soil sample P refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm SF stiff VS: very stiff H hard Fc friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no:
BH7
sheet 2 of 2

engineering log - borehole

office job no: S0425.1

client	GEOMARINE PTY LTD	note commenced	30 11 90
principal	WARRINGAH SHIRE COUNCIL	note completed	30 11 90
project	BEACH DEVELOPMENT DESIGN CRITERIA	logged by	SRM
borehole location	SEE DRAWING Bc	checked by	MAB

drill model and mounting	EDSON 3000 - TRUCK	slade	-90 DEG	RL Surface	10.3	m
hole diameter	100mm	pegging		datum	AHD	

method	depth (m)	soil type	moisture condition	consistency/density index	structure and additional observations
AD	0.0 - 0.5	SP SAND: medium grained light yellow brown little or no fines	M	MD	DUNE SAND, trace of shells
AD	0.5 - 1.0	SP SAND: medium to coarse grained light yellow orange & brown with some shell fragments Concretions observed in shell fragments Shell content increasing to >50% total	W	VD	DUNE SAND with indurated layers
Borehole BH7 Terminated at 10.20 m					

METHOD	SUPPORT	NOTES	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION	CONSISTENCY/DENSITY INDEX
AS auger screwing AD auger drilling F roller-torque W washbore CT cable tool HA hand auger D' orlatube *dit shown by suffix B blank dit V V dit T TC dit eg ADI	C casing M mud PENETRATION no resistance ranging to refusal WATER not measured water level water outflow water inflow	UIC undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N* SPT - sample recovered Nc SPT with solid cone V vane shear P pressuremeter Bt bulk sample R refusal	based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	VS very soft S soft F firm St stiff VSt very stiff H hard Fh friable VL very loose L loose MD medium dense D dense VD very dense

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borehole no
BH8
sheet 1 of 2

engineering log - borehole

office job no: S9425.1

client: GEOMARINE PTY LTD	hole commenced: 30/11/90
principal: WARRINGAH SHIRE COUNCIL	note completed: 30/11/90
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SRM
borehole location: SEE DRAWING B6	checked by: MAB

drill model and mounting: EDSON 3000 - TRUCK scope: -90 DEG RL Surface: 90 m
 note diameter: 100mm casing: section: AHD

method 1 2 3	penetration	support	water	notes samples, test, etc	RL	depth metres	graphic log	classification symbol	material soil type: plasticity or particle characteristics colour secondary and minor components	moisture condition	consistency density index etc	hand penetration metres		structure and additional observations
												mm	cm	
ADI		NHL				0		SP	SAND: fine to medium grained, light yellow brown, little or no fines	D				DUNE SAND with shell material
				1 . 2 . 1 N°=3		1				M				
						2								
				3 . 4 . 6 N°=10		3					MD			
						4								
				4 . 7 . 7 N°=14		5								
						6								
						7		SP	SAND: fine to medium grained, yellow brown orange, trace amounts of clay					
				4 . 6 . 6 N°=12		6			Layer of coarse sand & fine quartz gravel					
						7			Colour change to dark brown at about 7.0m					
				20 . N°=R		8			Incurated sand (?)		VD			SPT - 40 blows for 20mm
						9		SP	SAND: fine to coarse grained light yellow brown, trace of clay					Hole collapsing below water table

METHOD AS auger screwing AD auger drilling R roller/tricone W washbore CT cable tool HA hand auger DT dialtube *bit shown by suffix B blank bit V V bit T TC bit e.g. ADT	SUPPORT C casing M mud PENETRATION WATER • not measured water level water outflow water inflow	NOTES samples and tests U50 undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N° SPT + sample recovered NU SPT with solid cone V vane shear P pressuremeter Bs bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VS: very stiff H hard Fb friable VL very loose L loose MD medium dense D dense VD very dense
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borehole no
BH8
sheet 2 of 2

engineering log - borehole

office job no S9425 1

client: GEOMARINE PTY LTD	note commenced: 30.11.90
principal: WARRINGAH SHIRE COUNCIL	note completed: 30.11.90
project: BEACH DEVELOPMENT DESIGN CRITERIA	logged by: SRM
borehole location: SEE DRAWING B6	checked by: MAB

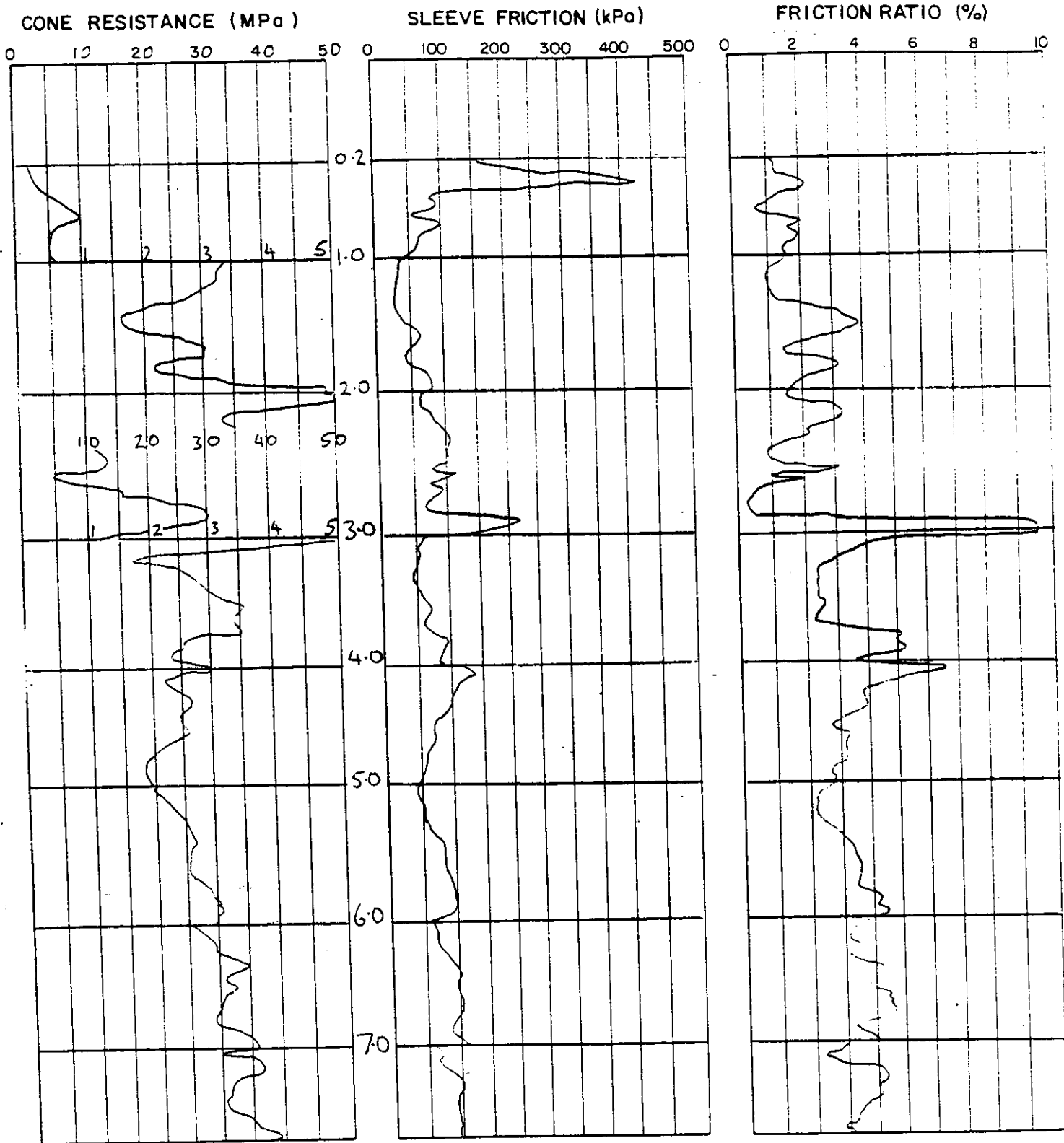
drill mode and mounting: EDSON 3000 - TRUCK	stop: -05 DEG	R.L Surface: 90 m.
hole diameter: 100mm	bearing:	datum: AHD

method	1 2 3 penetration	support	notes samples, test etc	PL	depth metres	graphic log	classificational symbol	material soil type; plasticity or particle characteristics colour secondary and minor components	moisture condition	consistency density index	hand kPa electronic meter				structure and additional observations
											net	total	tip	bar	
ADT					0		SP	SAND fine to coarse grained, light yellow brown trace of clay	W	VD					
					-1										
					-2			Borehole BH8 Terminated at 10.25 m							
					-3										
					-4										
					-5										
					-6										

METHOD AS auger screwing* AD auger drilling** R roller tricone W washbore CT cable tool HA hand auger DT diatube *bit shown by suffix P plain bit V V bit T TC bit e.g. ADT	SUPPORT C casing M mud PENETRATION no resistance ranging 1c refusal WATER not measured water level water outflow water inflow	NOTES samples and tests US undisturbed sample 50 mm diameter D disturbed sample N standard penetration test N* SPT - sample recovered Nc SPT with solid cone V vane shear P pressuremeter Bc bulk sample R refusal	CLASSIFICATION SYMBOLS AND SOIL DESCRIPTION based on unified classification system MOISTURE D dry M moist W wet Wp plastic limit	CONSISTENCY/DENSITY INDEX VS very soft S soft F firm St stiff VSt very stiff H hard Fb friable VL very loose L loose MD medium dense D dense VD very dense
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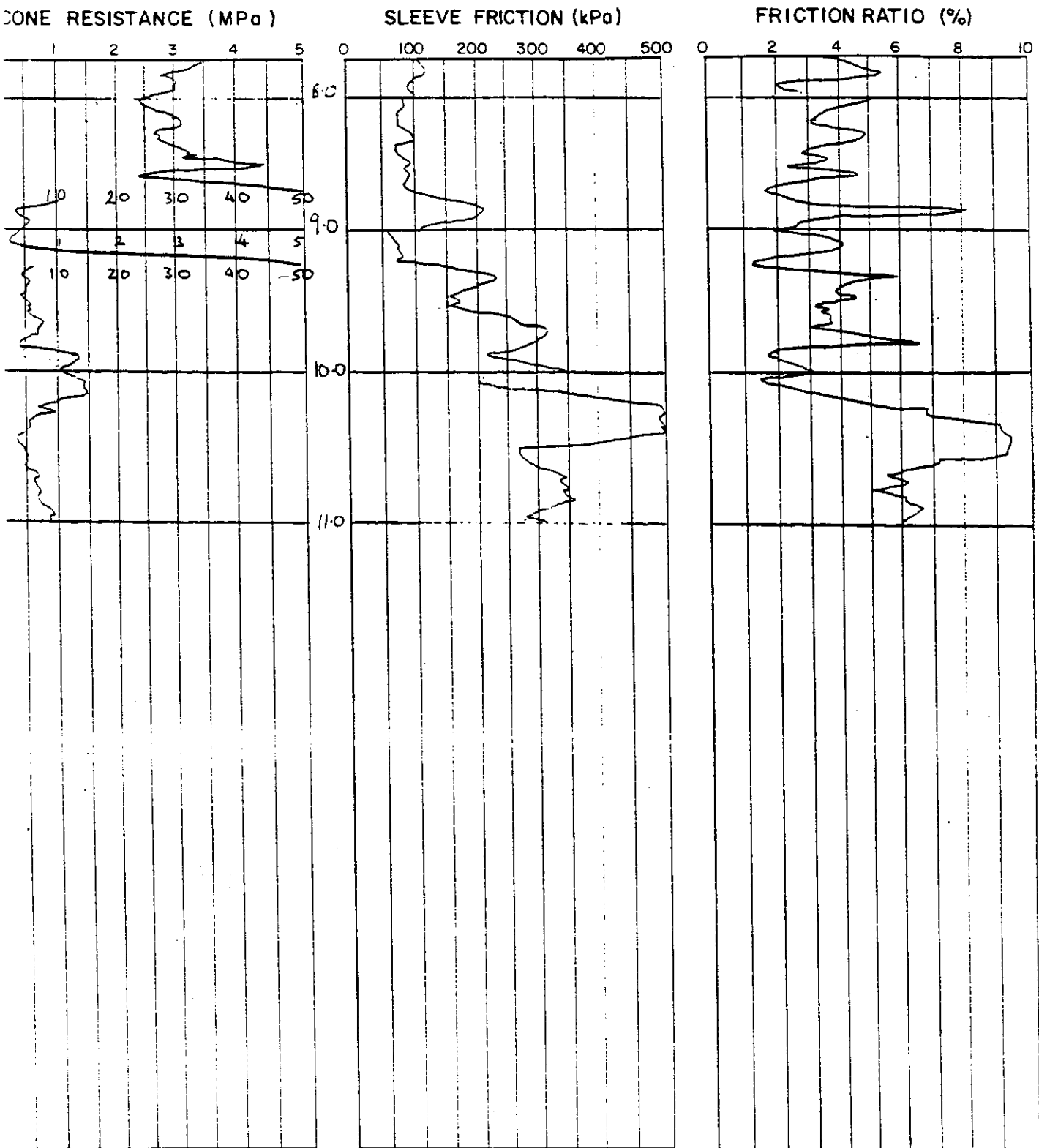
TEST STARTED - 0.2



TEST TERMINATED-

ELECTRIC FRICTION CONE PENETROMETER TEST - EFCPI

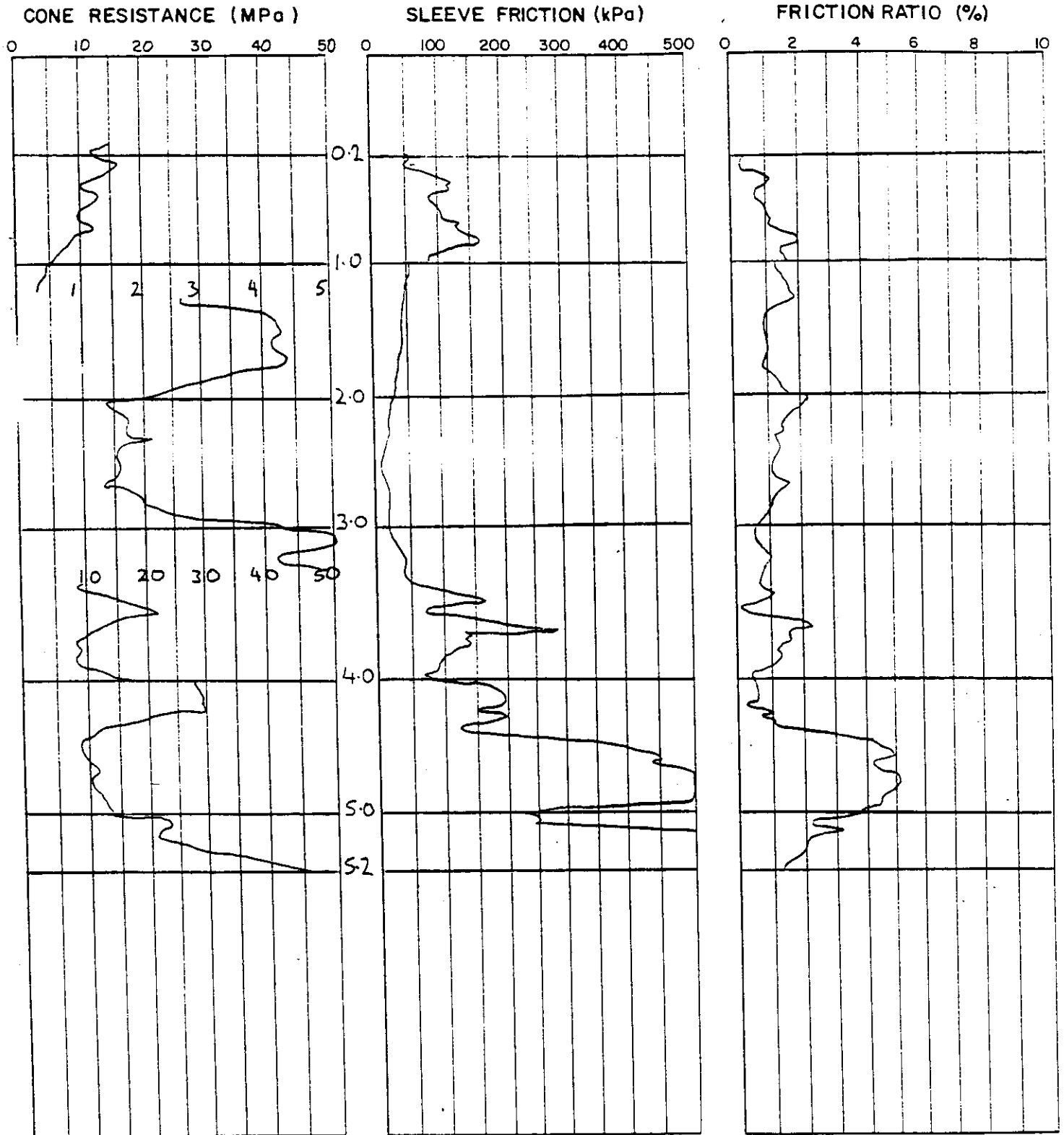
TEST STARTED -



TEST TERMINATED - 11.0

ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP1

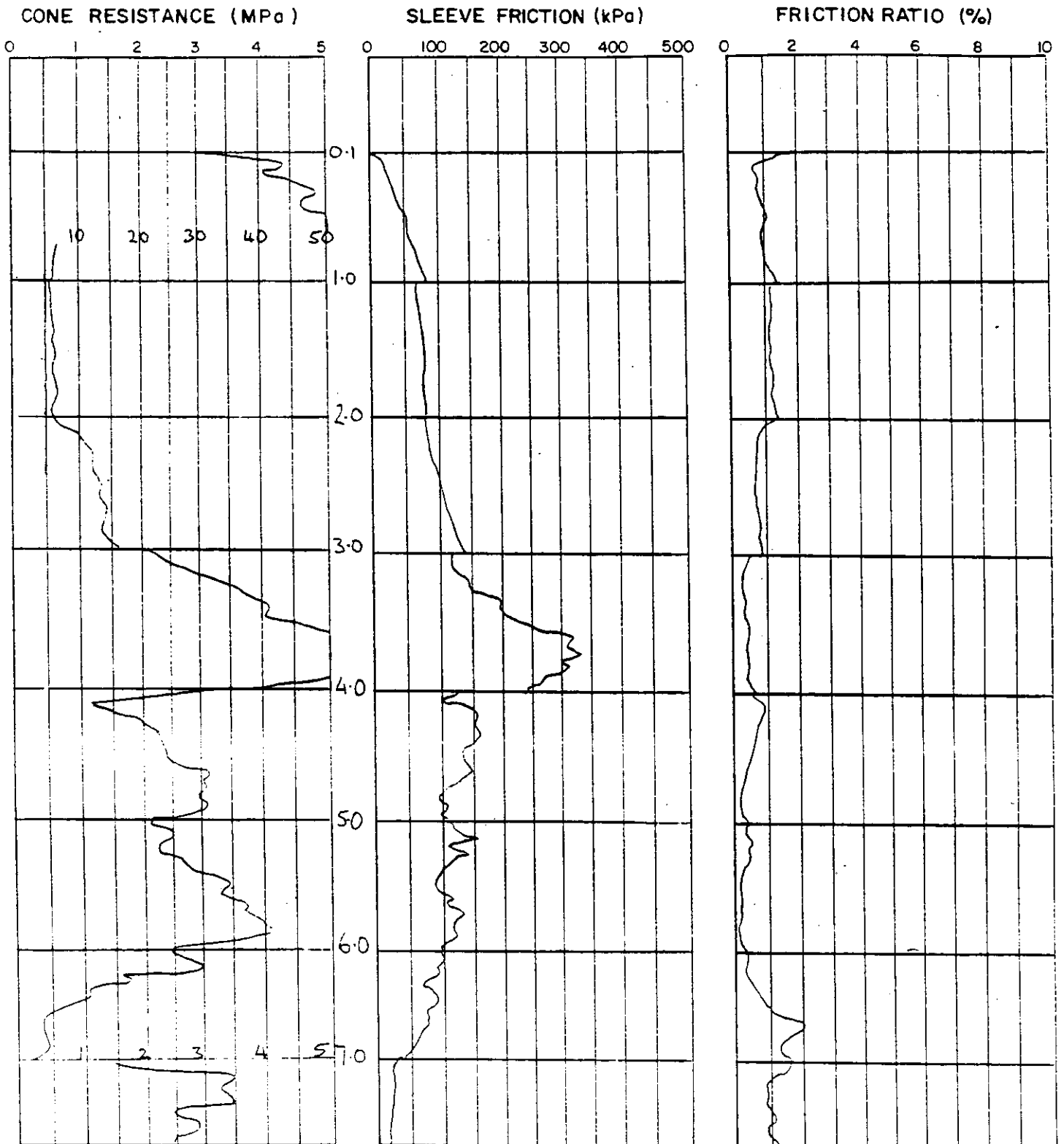
TEST STARTED - 0.2



TEST TERMINATED - 5.2

ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP2

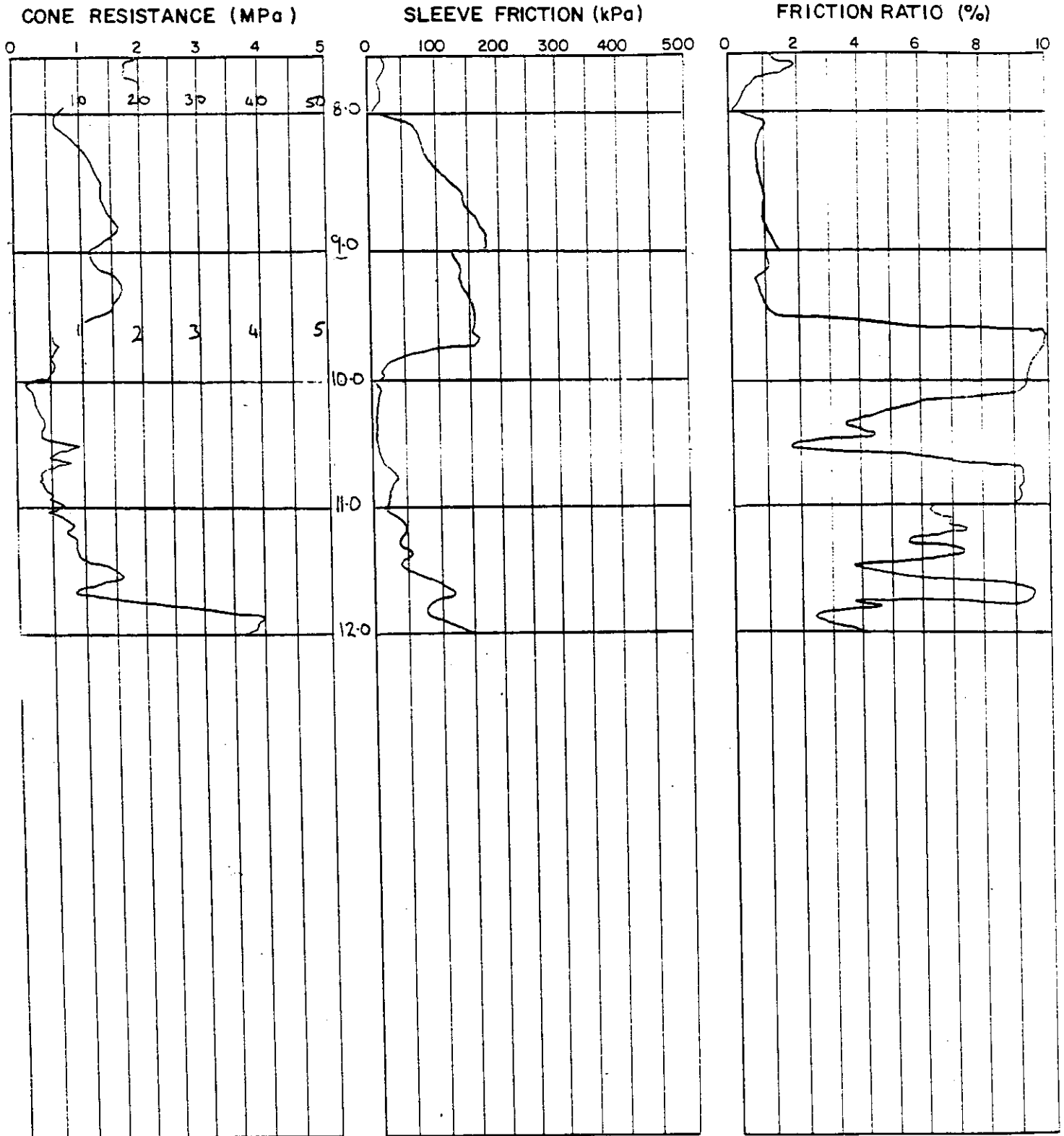
TEST STARTED - 0.1



TEST TERMINATED-

ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP3

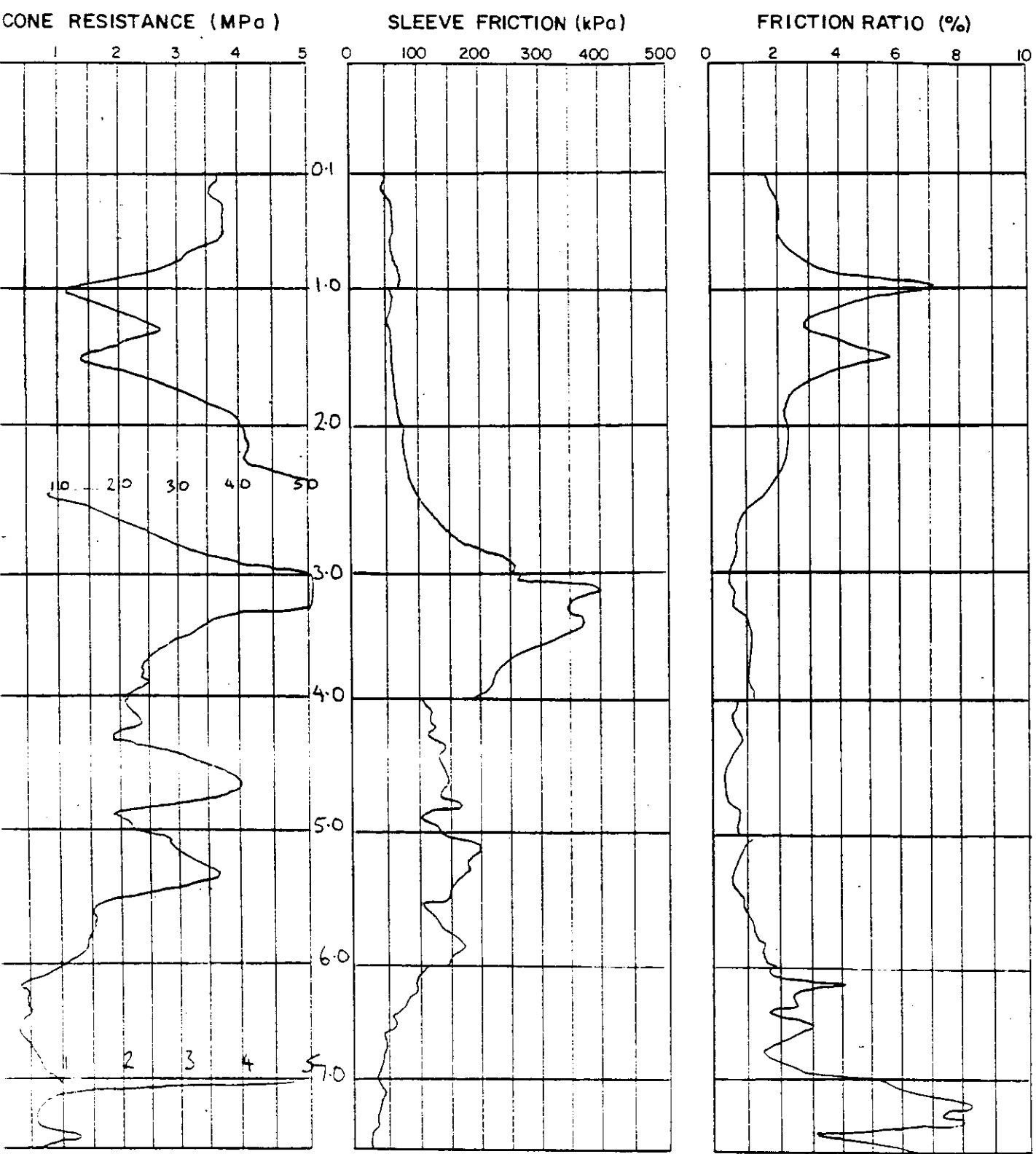
TEST STARTED -



TEST TERMINATED - 12.0m

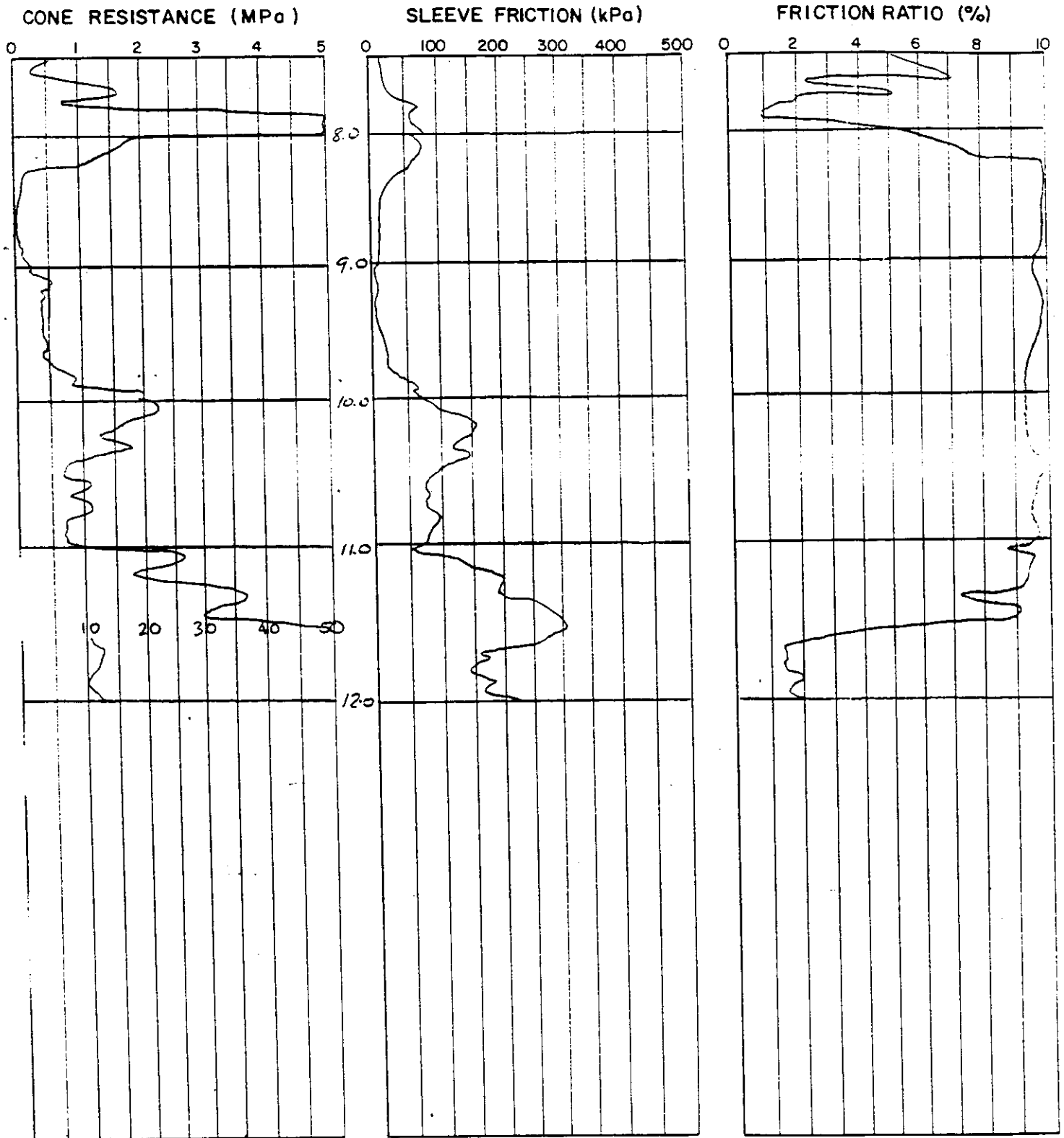
ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP3

TEST STARTED - 0.1



TEST TERMINATED -

TEST STARTED -



TEST TERMINATED - 12.0

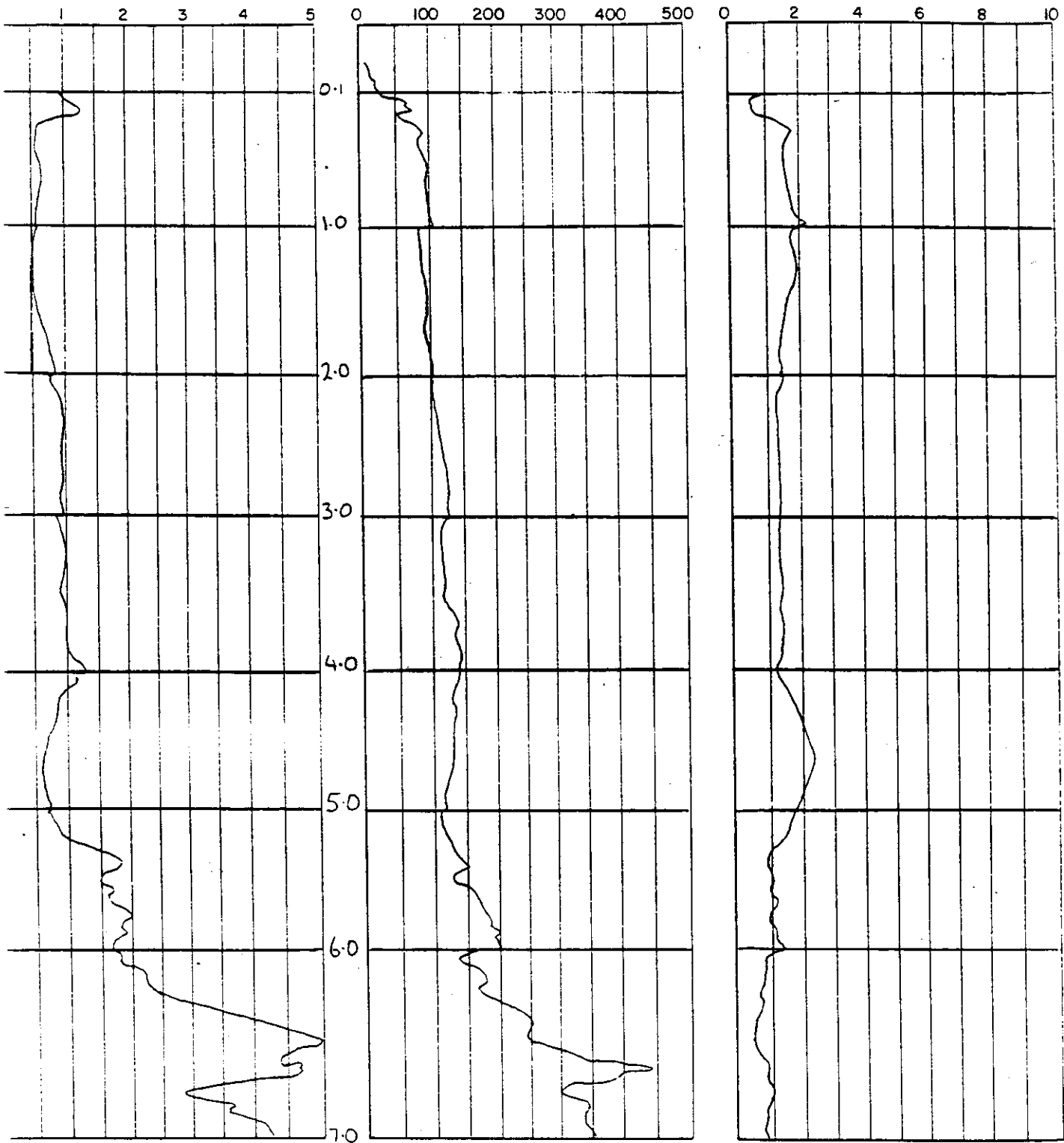
ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP3B

TEST STARTED - 0.1

CONE RESISTANCE (MPa)

SLEEVE FRICTION (kPa)

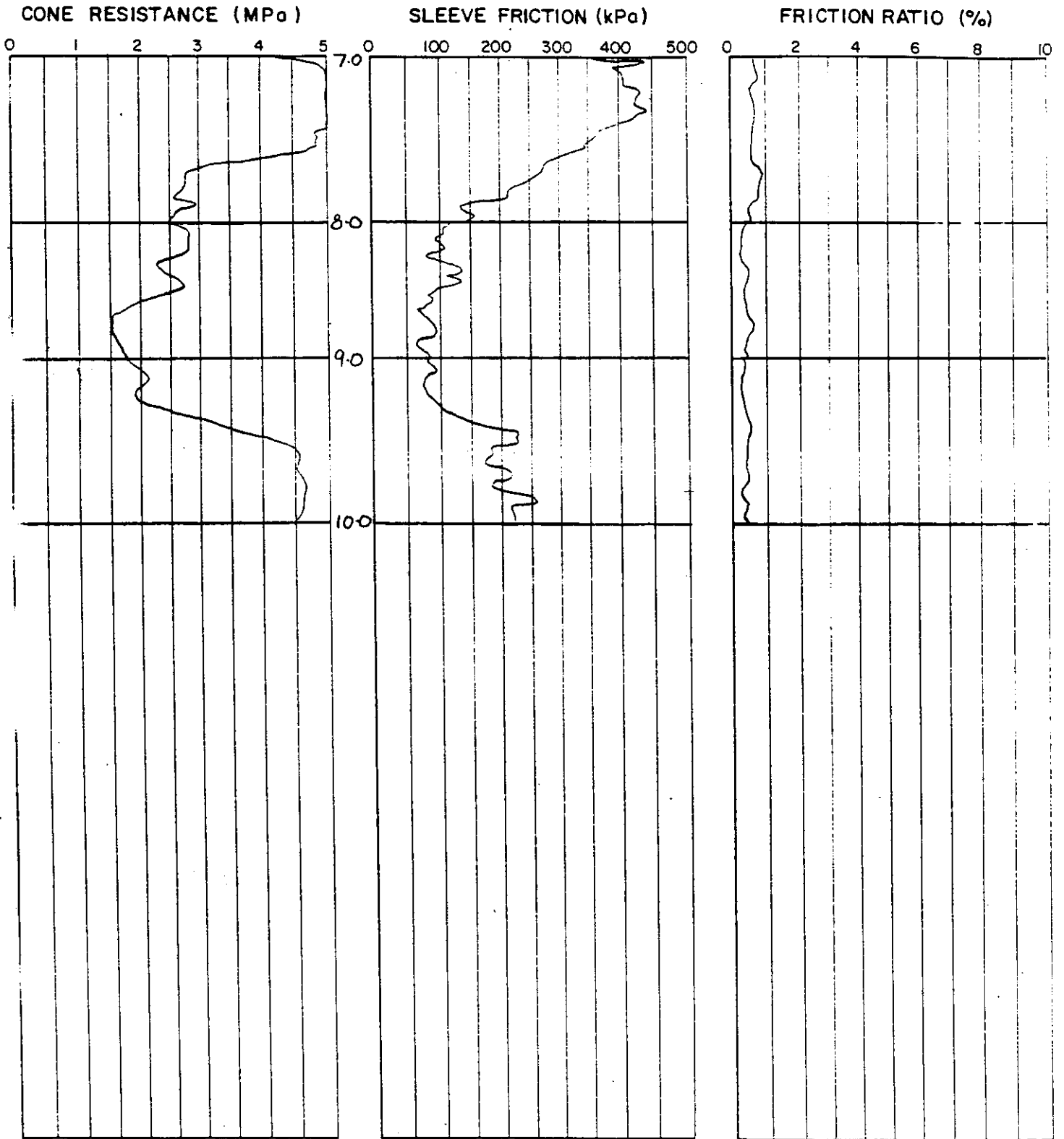
FRICTION RATIO (%)



TEST TERMINATED -

ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP4

TEST STARTED -



TEST TERMINATED -

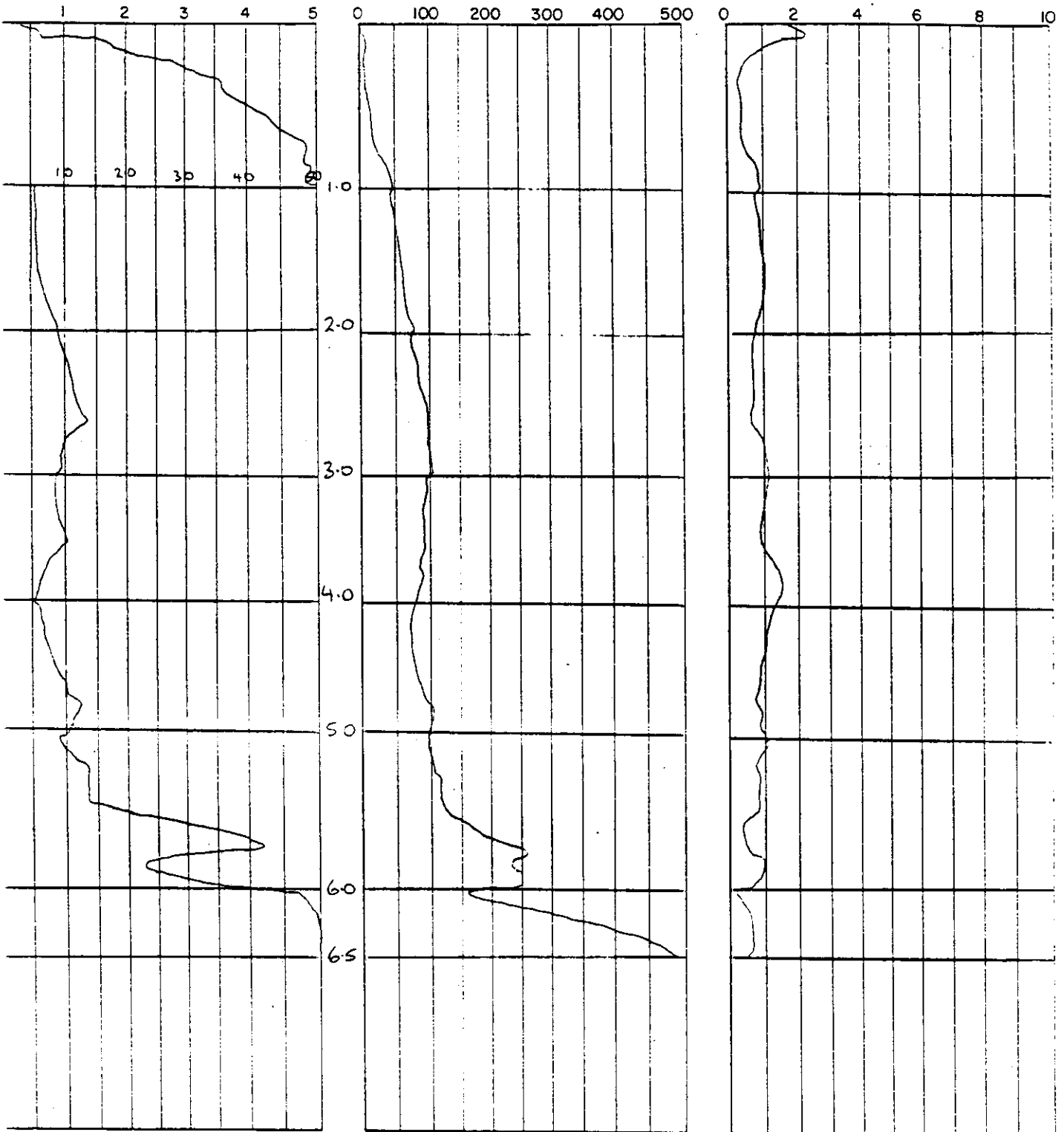
ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP4

TEST STARTED - 0.1m

CONE RESISTANCE (MPa)

SLEEVE FRICTION (kPa)

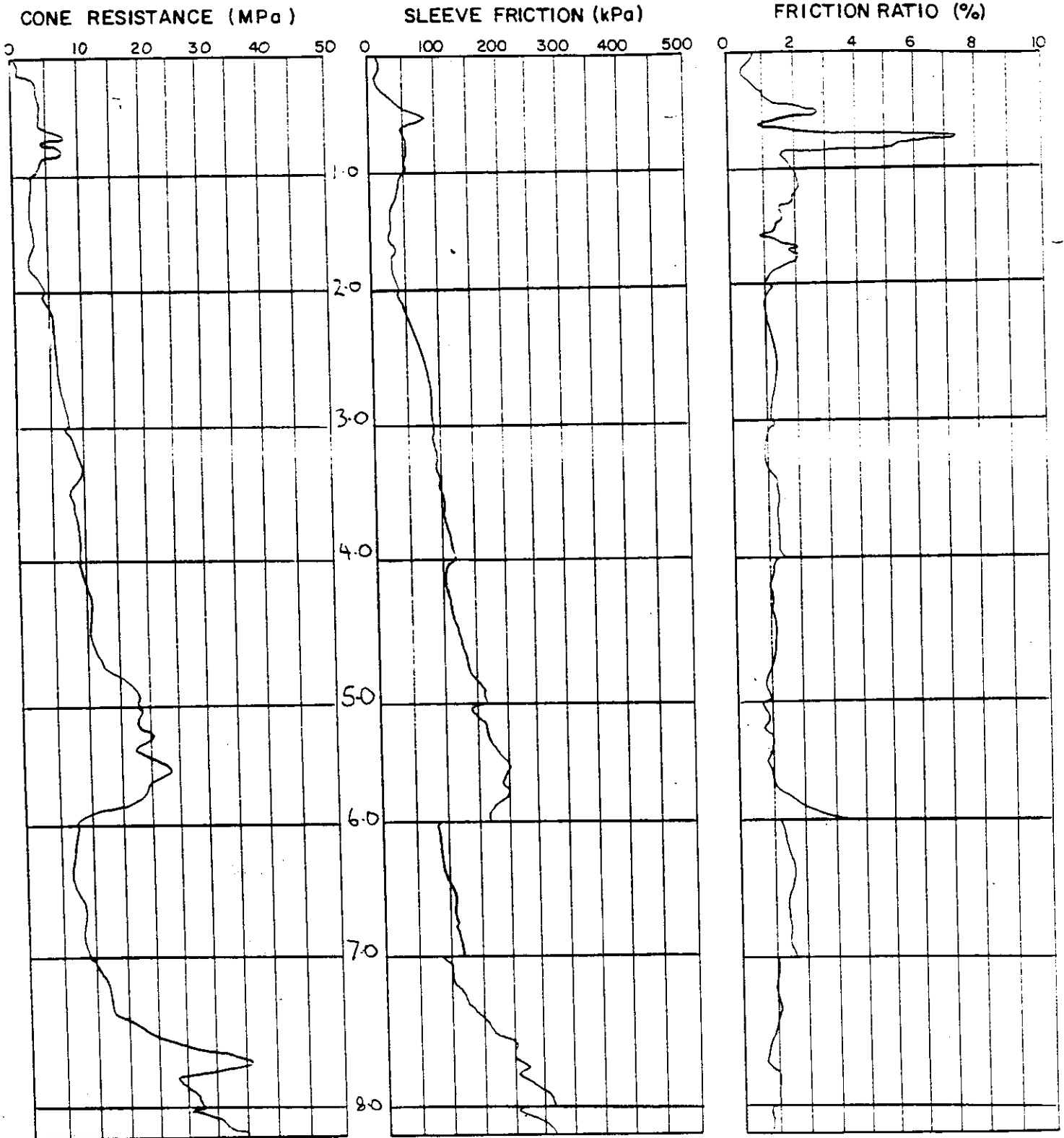
FRICTION RATIO (%)



TEST TERMINATED - 6.5m

ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP5A

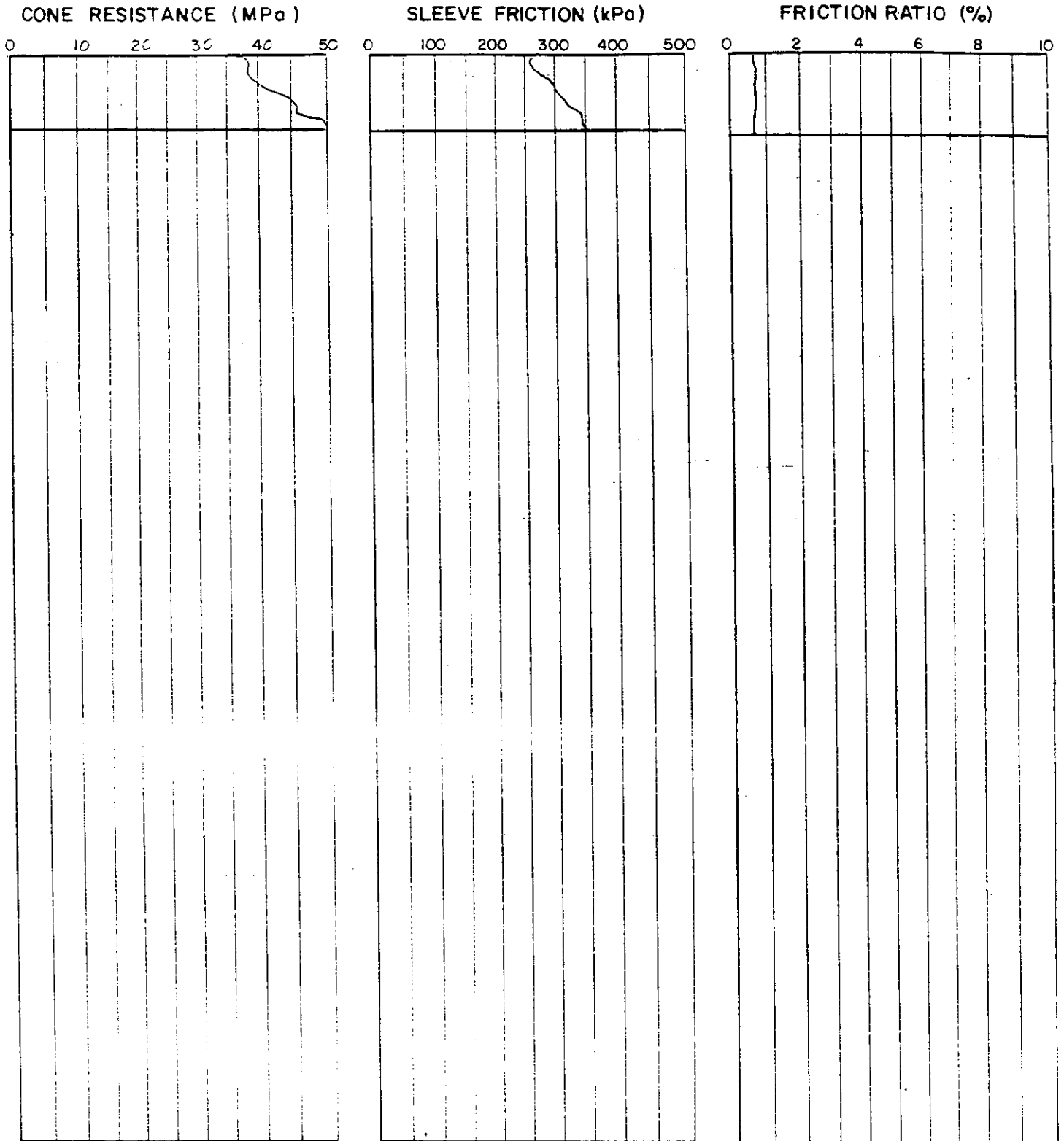
TEST STARTED - 01



TEST TERMINATED-

ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP6A

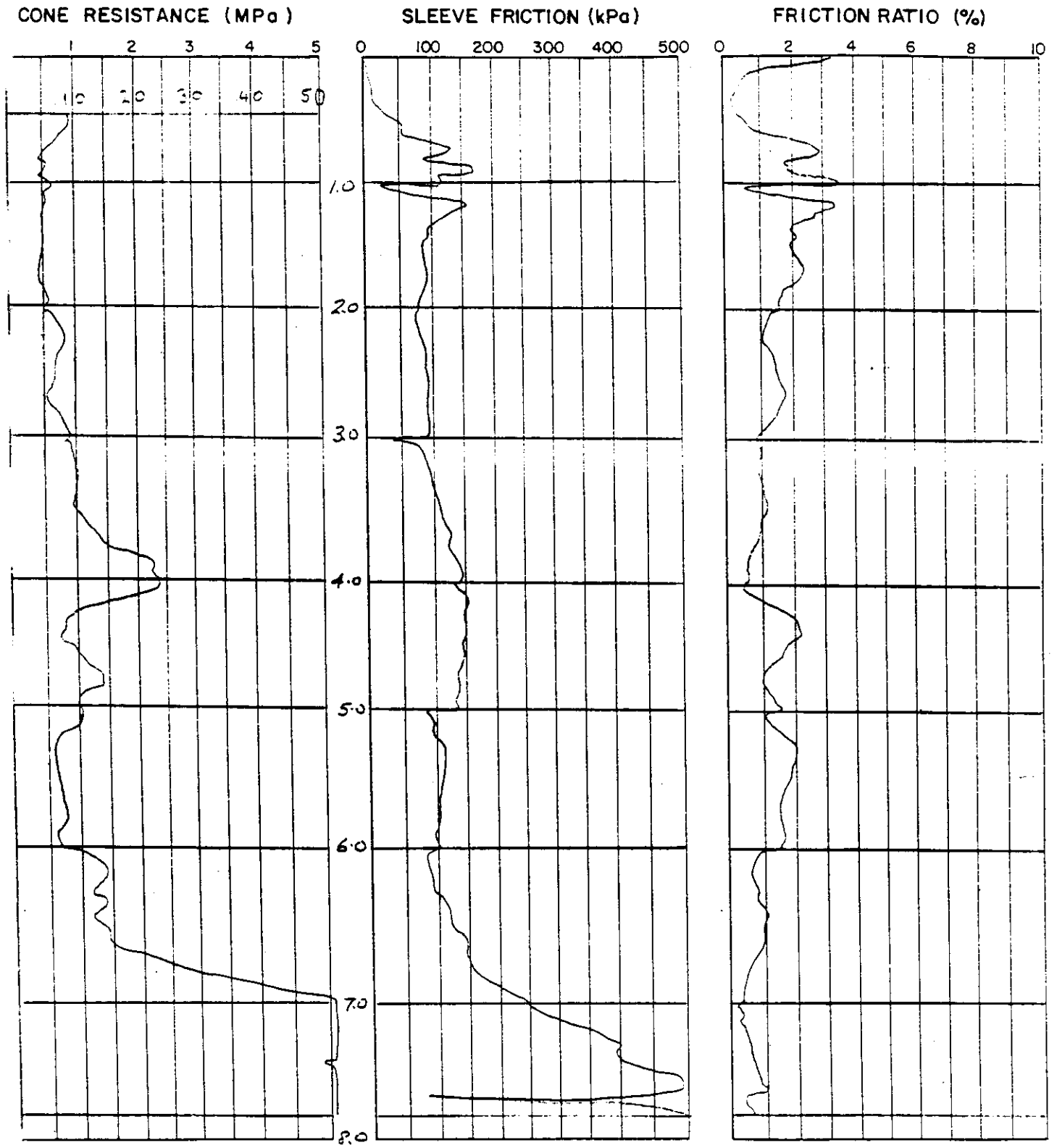
TEST STARTED -



TEST TERMINATED - 8.7m

ELECTRIC FRICTION CONE PENETROMETER TEST - EFCP6A

TEST STARTED - 0.1m



TEST TERMINATED - 7.7m

Appendix C

Laboratory Testing Data



borehole:
BH3
sheet 1 of 1

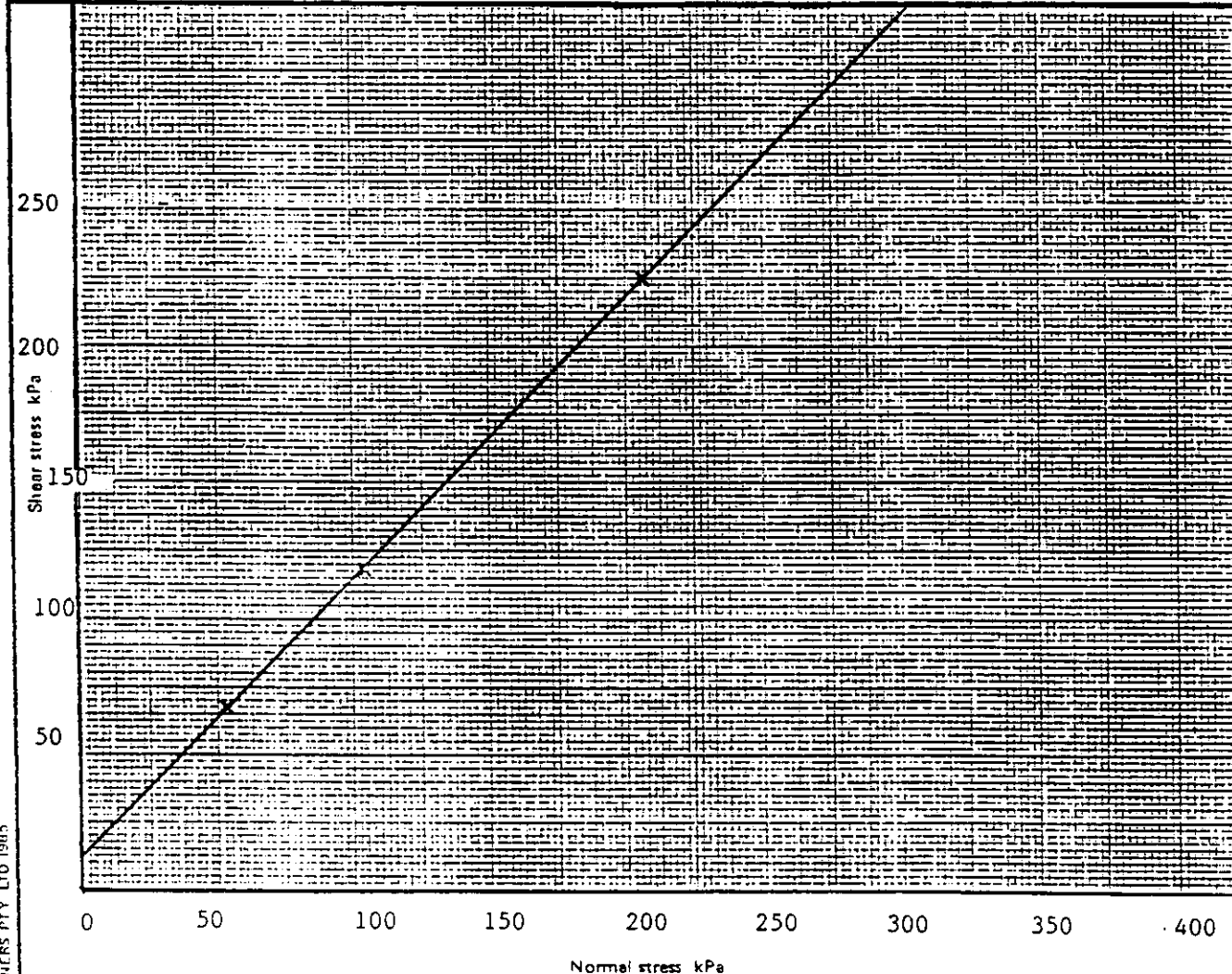
direct shear test

office: SYDNEY

client: GEOMARINE PTY LTD	job no: S9425/1
principal: WARRINGAH SHIRE COUNCIL	date: 21-12-90
project: BEACH DEVELOPMENT DESIGN CRITERIA	tested by: GC
location: NARRABEEN TO COLLAROY BEACH	checked by: JR

depth: 4.10 - 4.55m	sample type: 3 Single Stage	sample size: 71 x 71 x 34mm
type of test: consolidated Drained	remoulded	

material classification: (SC/SP) SAND, fine to coarse, yellow-brown, some fines of low plasticity, trace of fine to medium gravel, trace of shell fragments.



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Initial moisture content:	17.8	%
Initial dry density:	1.62	t/m ³
Cohesion C:	12.5	kPa
Angle of friction ϕ :	45.5	$^{\circ}$

remarks:

failure criteria: Peak Shear Stress



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8 1 91
 Authorised Signature *James Russell*



job no:
S9425/1
sheet 1 of 1

direct shear test

office: SYDNEY

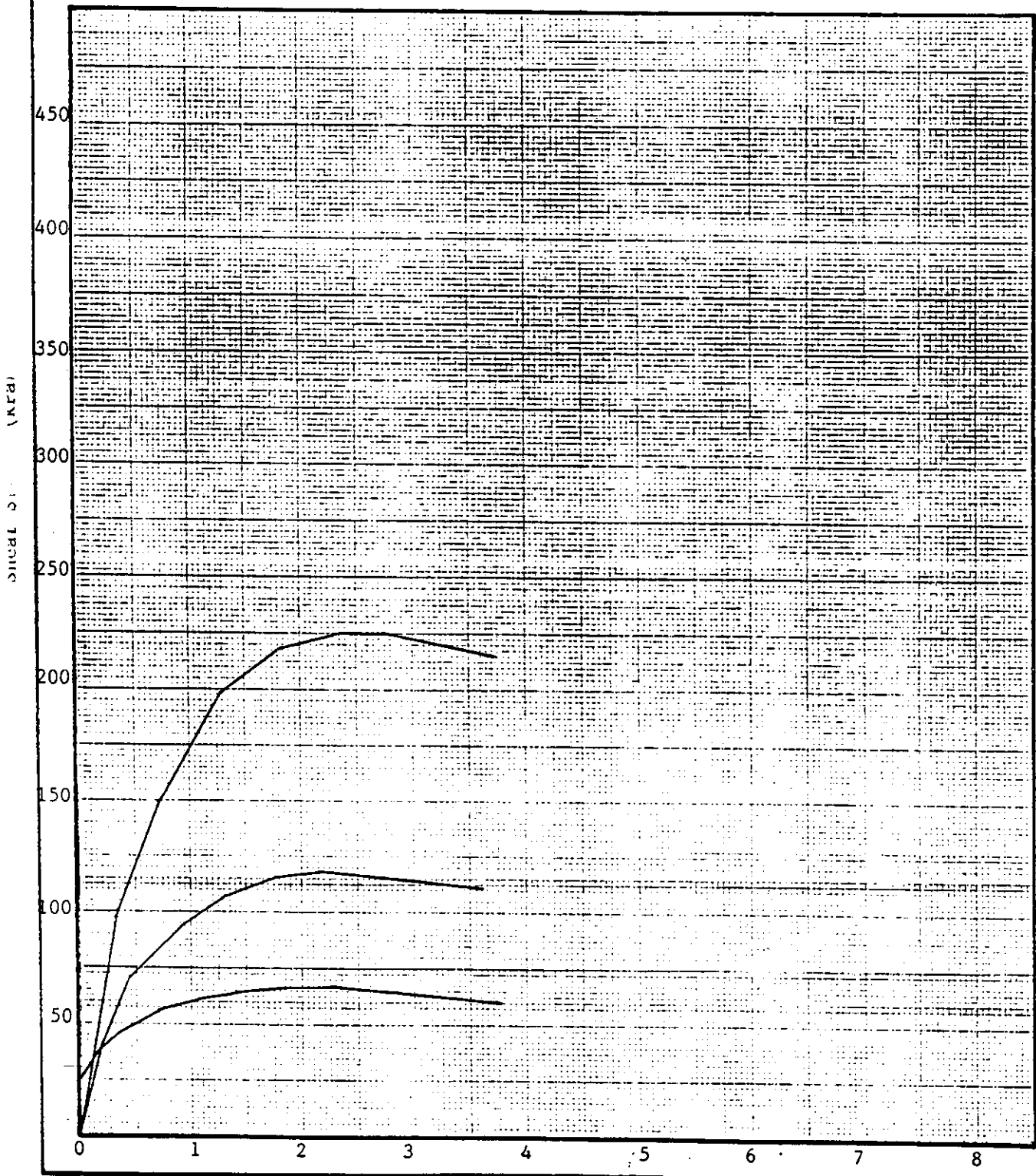
date: 4-1-91

tested by: GC

checked by: JR

client: GEOMARINE PTY LTD
principal: WARRINGAH SHIRE COUNCIL
project: BEACH DEVELOPMENT DESIGN CRITERIA
location: NARRABEEN TO COLLAROY BEACH

borehole: BH3 depth 4.1 - 4.55m stress range: - kPa



Relative Deflection (mm)



borehole:
BH5
sheet 1 of 1

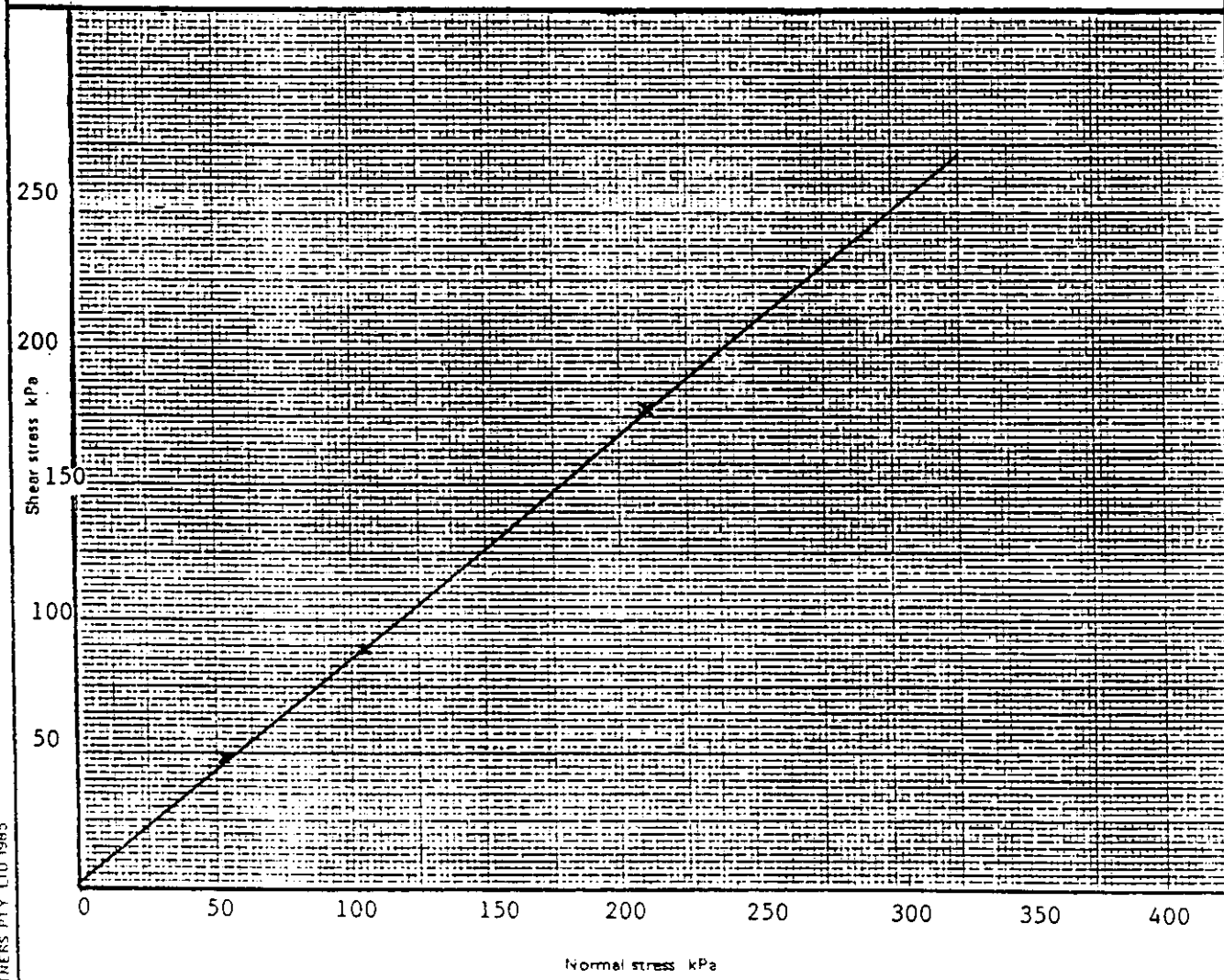
direct shear test

office: SYDNEY

client: GEOMARINE PTY LTD	job no: S9425/1
principal: WARRINGAH SHIRE COUNCIL	date: 21-12-90
project: BEACH DEVELOPMENT DESIGN CRITERIA	tested by: GC
location: NARRABEEN TO COLLAROY BEACH	checked by: JR

depth: 1.10 - 1.55m sample type: 3 single stage - remoulded sample size: 71 x 71 x 34mm
 type of test: Consolidated Drained

material classification: (SP) SAND, fine to coarse, yellow-brown, trace of fines.



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Initial moisture content	2.6	%
Initial dry density	1.54	t/m ³
Cohesion C'	2	kPa
Angle of friction ϕ	39.5	°

remarks:
failure criteria: Peak Shear Stress



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8 1 91
 Authorised Signature *[Handwritten Signature]*



job no:
S9425/1
sheet 1 of 1

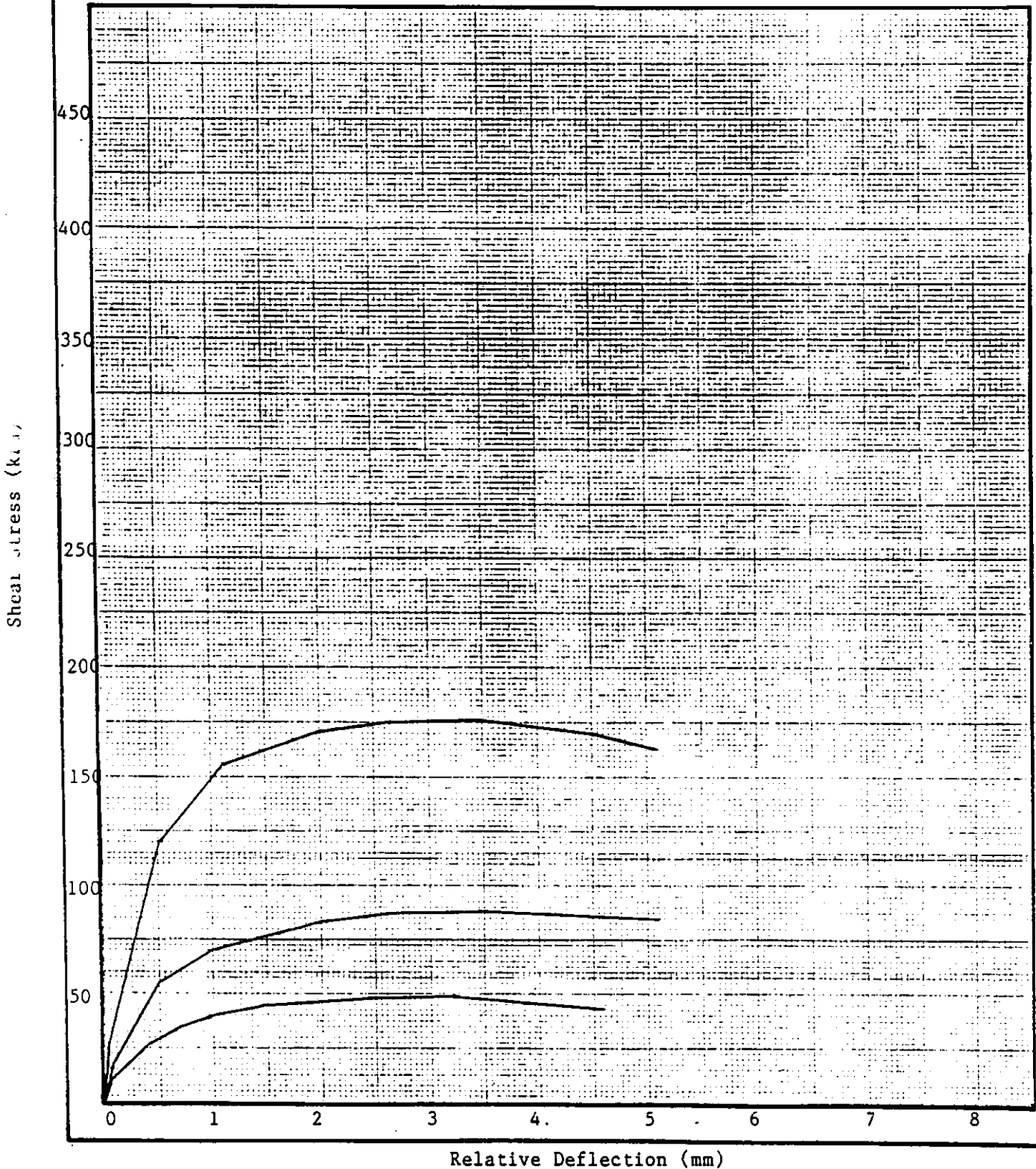
direct shear test

office: SYDNEY

client: GEOMARINE PTY LTD
principal: WARRINGAH SHIRE COUNCIL
project: BEACH DEVELOPMENT DESIGN CRITERIA
location: NARRABEEN TO COLLAROY BEACH

date: 4-1-91
tested by: GC
checked by: JR

borehole: BH5 depth 1.10 - 1.55m stress range: - kPa





borehole
BH7
sheet 1 of 1

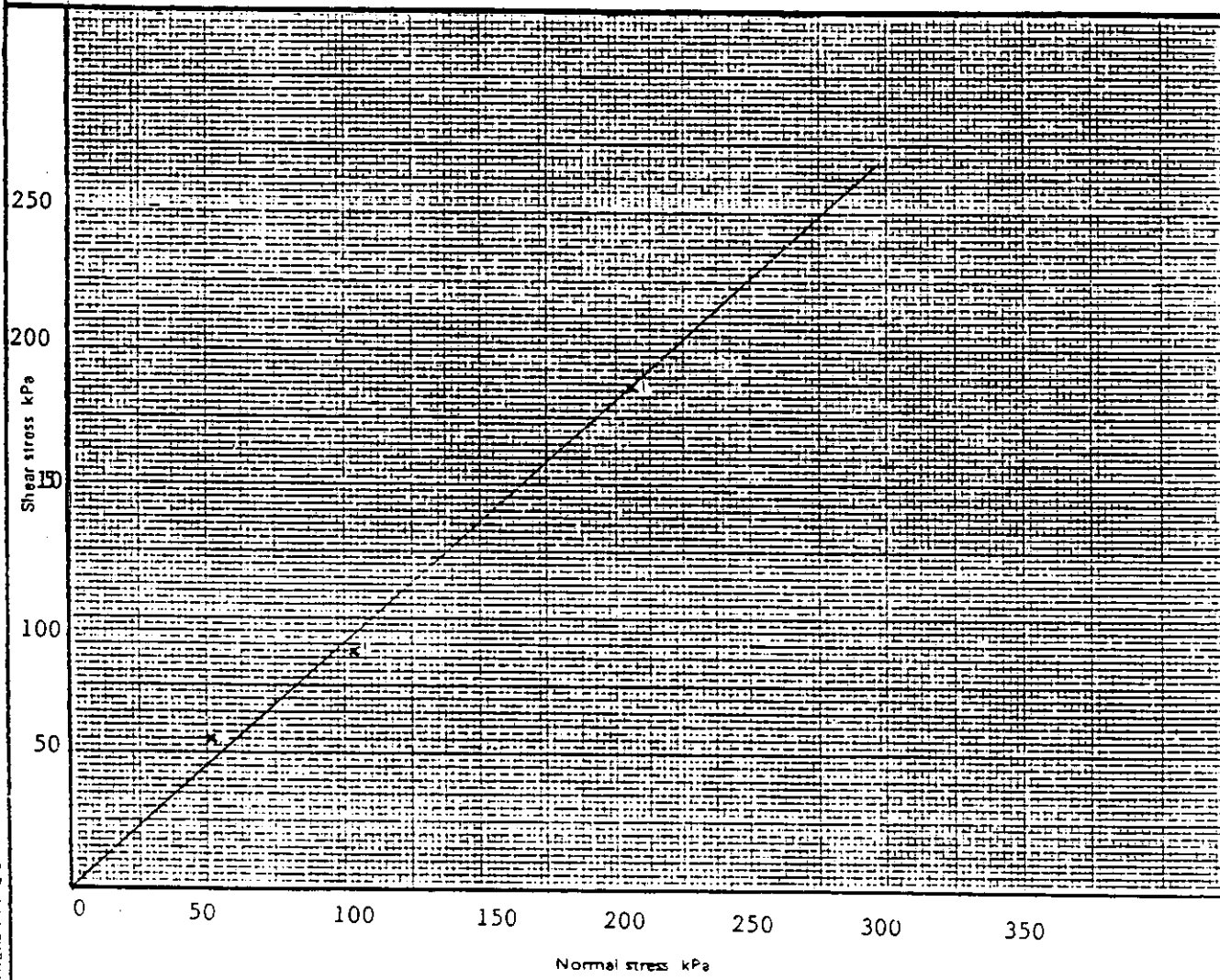
direct shear test

office: SYDNEY

client: GOEMARINE PTY LTD	job no: S9425/1
principal: WARRINGAH SHIRE COUNCIL	date: 20/12/90
project: BEACH DEVELOPMENT DESIGN CRITERIA	tested by: GC
location: NARRABEEN TO COLLAROY BEACH	checked by: JR

depth: 5.60 - 6.05m	sample type: 3 single stage - remoulded	sample size: 71x71x34mm
type of test: Consolidated Drained		

material classification: (SP) SAND - fine to coarse, yellow brown, trace of fines



Initial moisture content	2.6	%
Initial dry density	1.58	t/m ³
Cohesion C'	0	kPa
Angle of friction ϕ	42	°

remarks:
failure criteria: Peak Shear Stress

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8 1 91
Authorised Signature *James Russell*



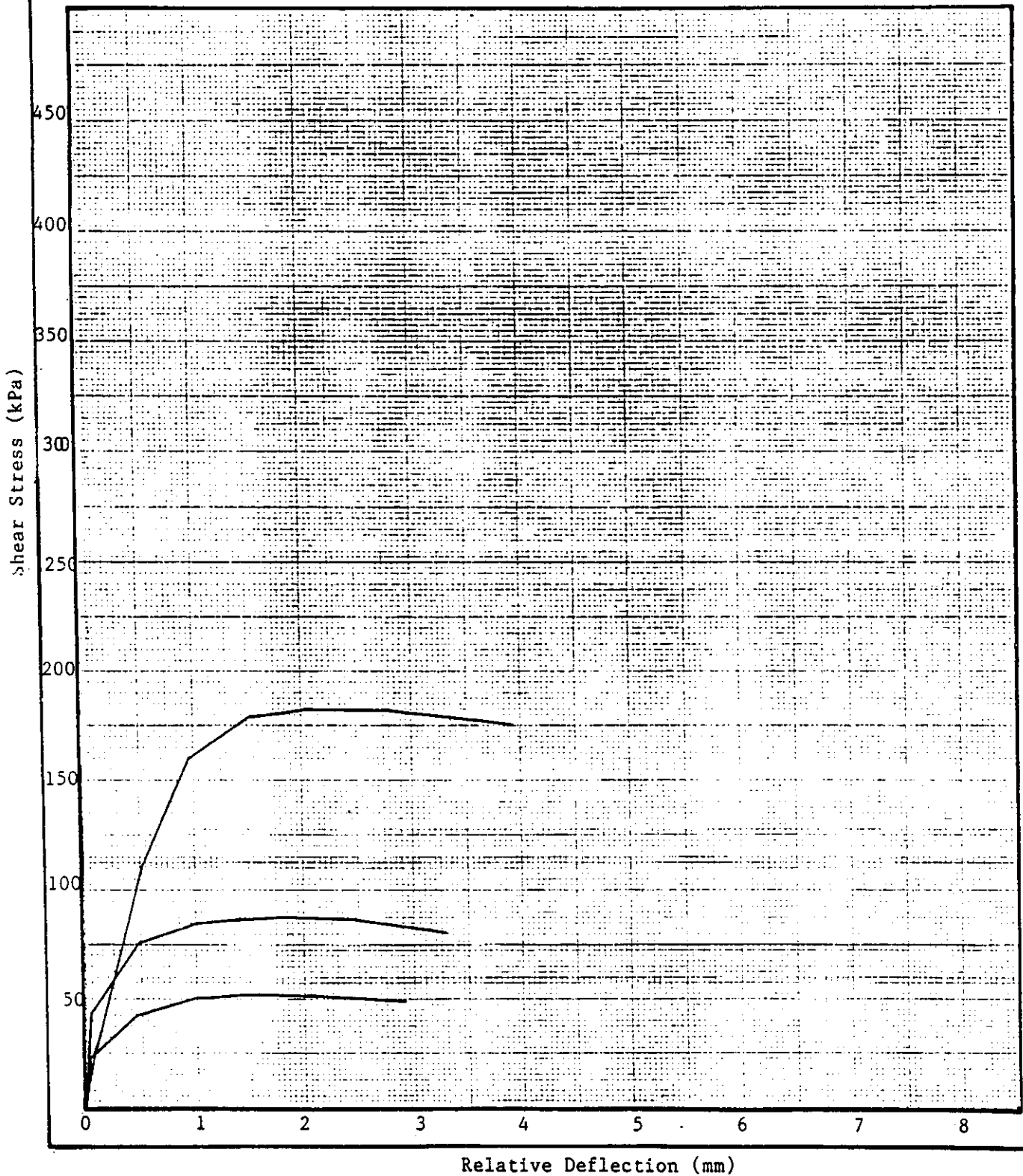
job no:
S9425/1
sheet 1 of 1

direct shear test

office: SYDNEY

client:	GEOMARINE PTY LTD	date:	4-1-91
principal:	WARRINGAH SHIRE COUNCIL	tested by:	GC
project:	BEACH DEVELOPMENT DESIGN CRITERIA	checked by:	JR
location:	NARRABEEN TO COLLAROY BEACH		

borehole:	BH7	depth	5.6 - 6.05m	stress range:	-	kPa
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borehole: BHS
sheet 1 of 1

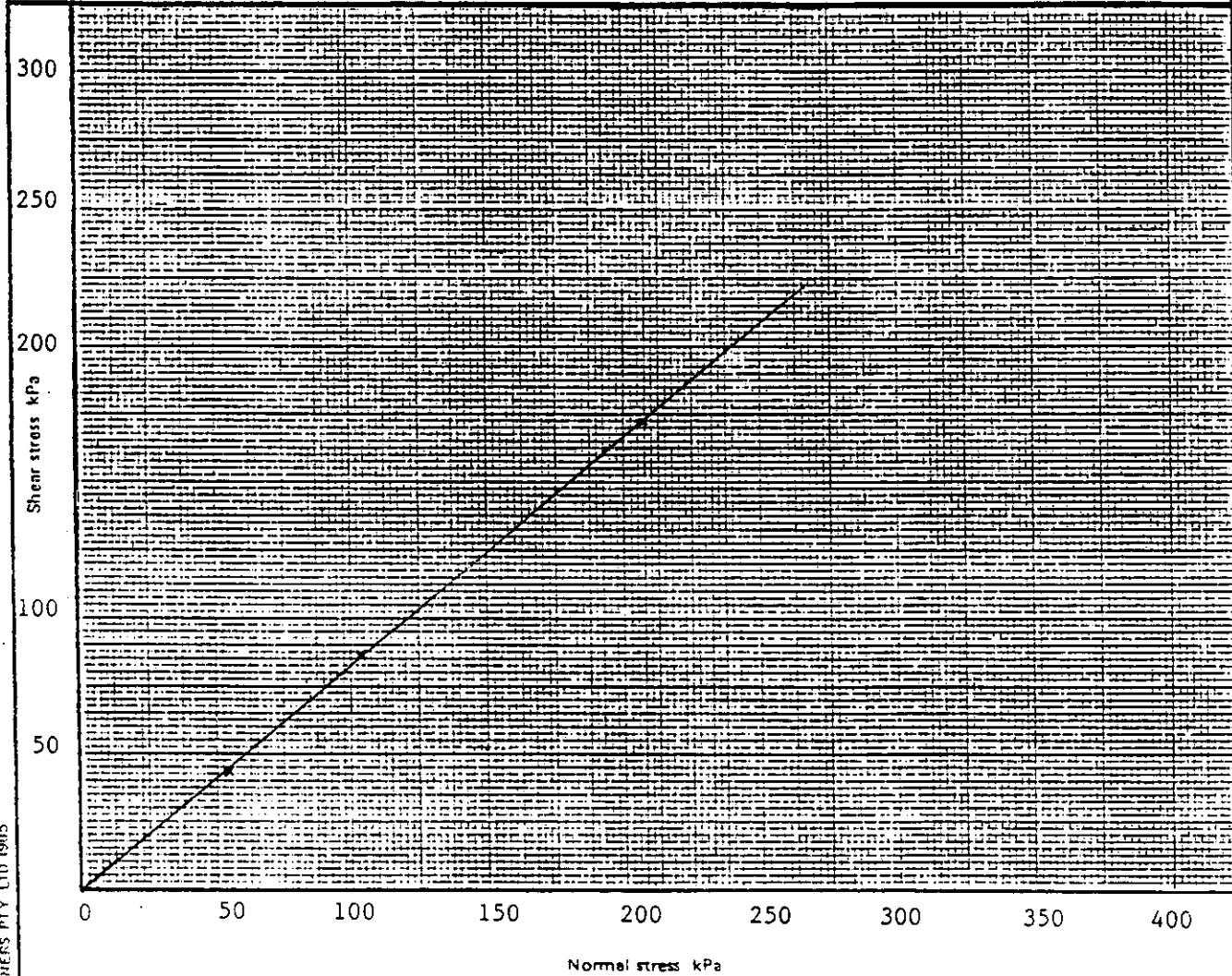
direct shear test

office: SYDNEY

client: GEOMARINE PTY LTD	job no: S9425/1
principal: WARRINGAH SHIRE COUNCIL	date: 20/12/90
project: BEACH DEVELOPMENT DESIGN CRITERIA	tested by: GC
location: NARRABEEN TO COLLAROY BEACH	checked by: JR

depth: 1.00 -1.45m sample type: 3 Singles Stage- sample size: 71x71x33mm
type of test: Consolidated Drained Test remoulded

material classification: (SP) SAND - fine to coarse, yellow brown, trace of fines



Initial moisture content 2.7 %
Initial dry density 1.57 t/m³
Cohesion C' 0 kPa
Angle of friction ϕ 4.0 °

remarks:
failure criteria: Peak Shear Stress

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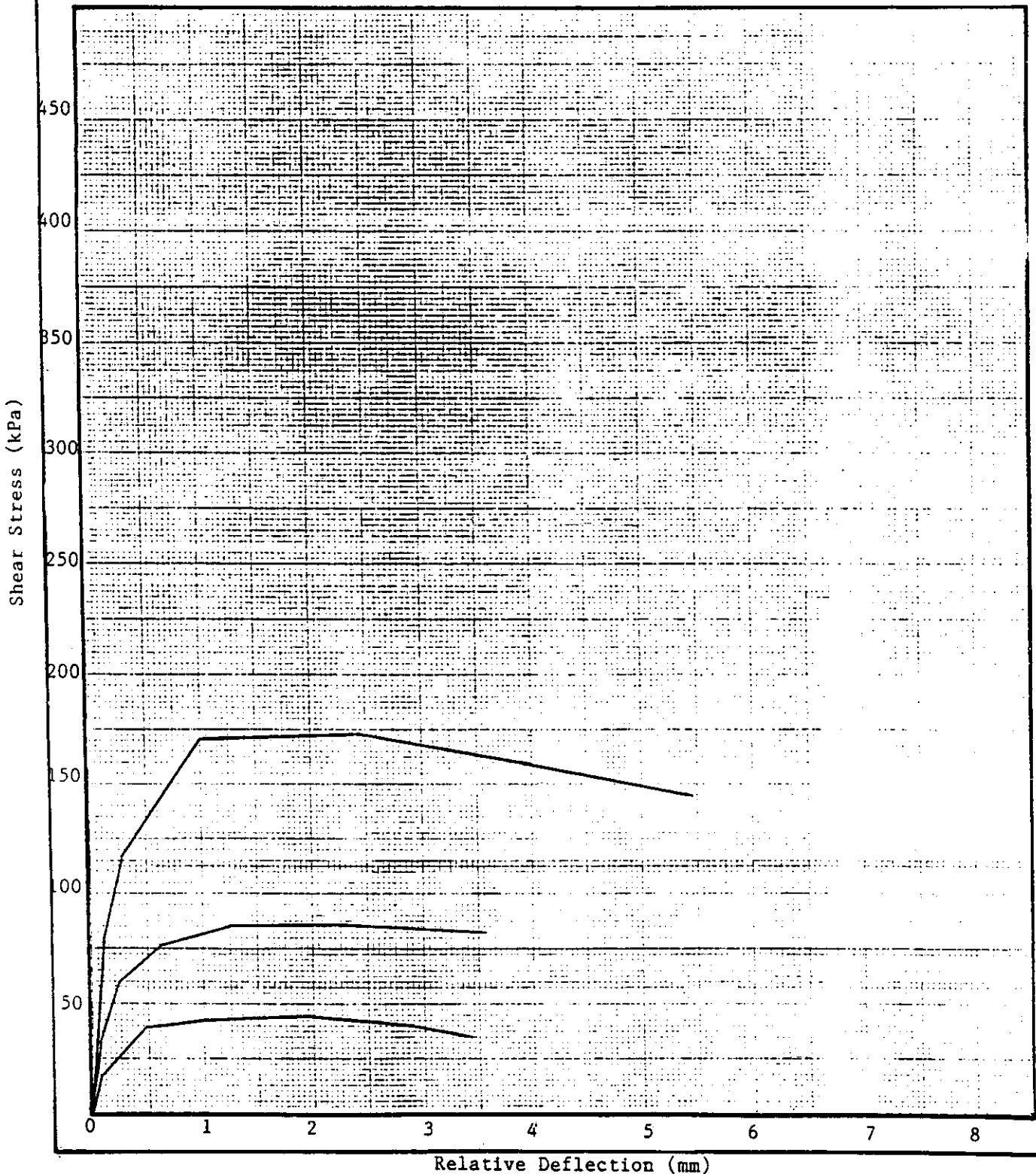
job no:
S9425/1
sheet 1 of 1

direct shear test

office: SYDNEY

client:	GEOMARINE PTY LTD	date:	4-1-90
principal:	WARRINGAH SHIRE COUNCIL	tested by:	GC
project:	BEACH DEVELOPMENT DESIGN CRITERIA	checked by:	JR
location:	NARRABEEN TO COLLAROY BEACH		

borehole:	BH8	depth	1.00 - 1.45m	stress range:	-	kPa
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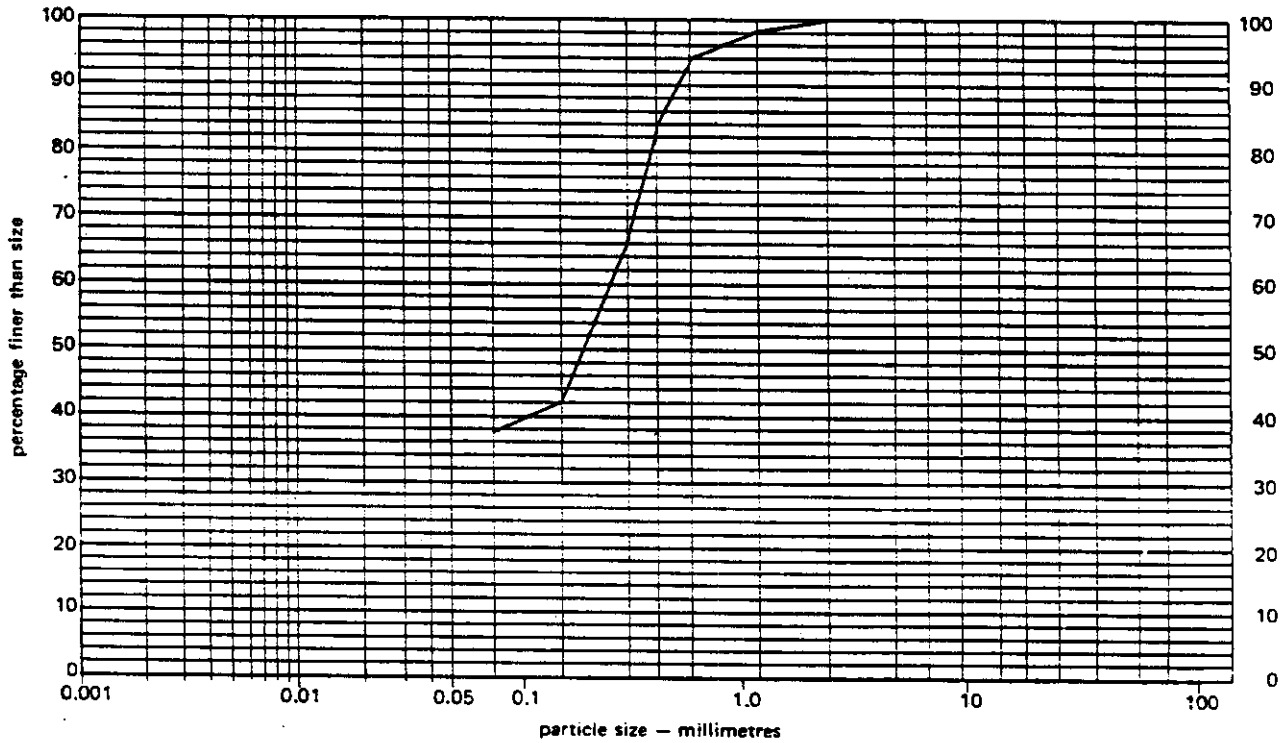
borehole no
 BH1
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC
sample identification	BH1	depth	2.6-3.05m
test procedure	AS1289 C6.1 - 1977		

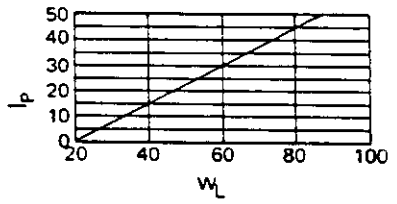
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
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0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%
plastic limit	%
plasticity index	%
linear shrinkage	%
particle density γ_m^a	
natural moisture	%



classification
 (SC) Clayey SAND- fine to coarse, yellow brown, fines of low to medium plasticity.

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borehole no
 BH2
 sheet 1 of 1

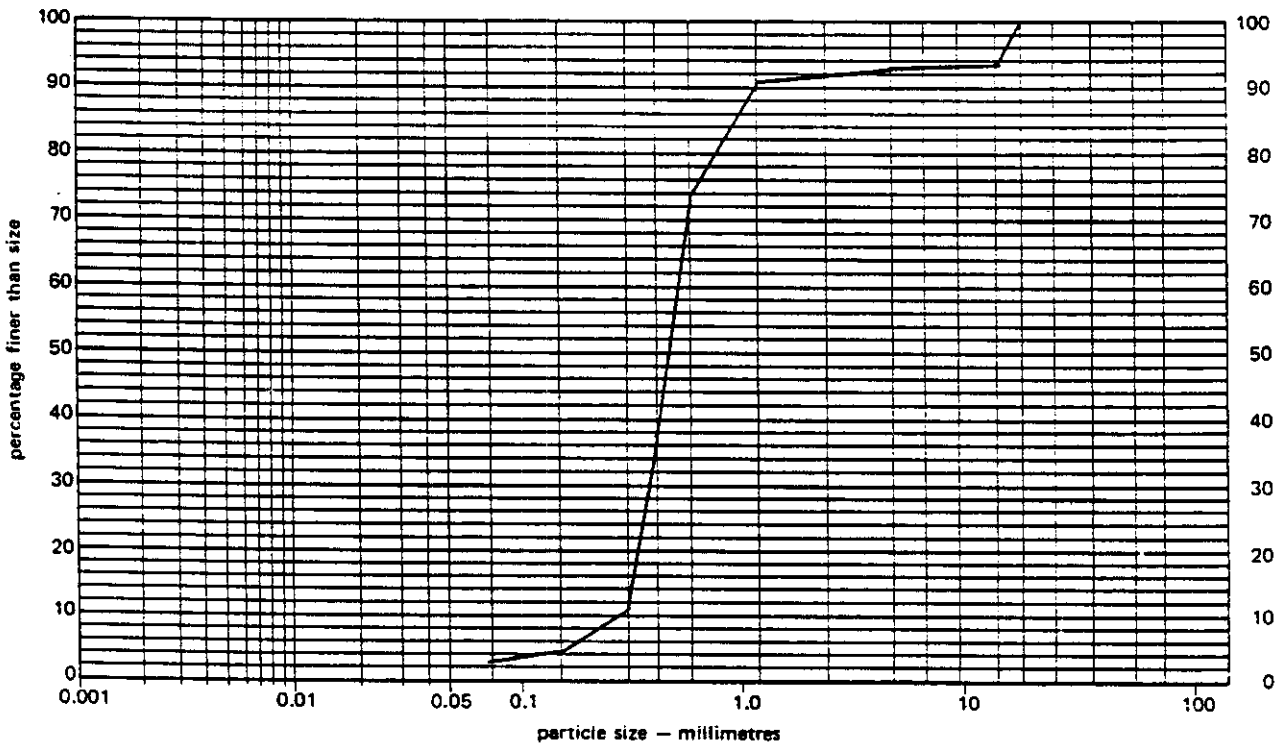
particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC

sample identification	BH2	depth	2.0-2.45m
test procedure	AS1289 C6.1 - 1977		

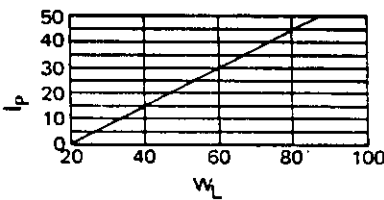
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
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	0.002	0.06			2.0			60			
clay	silt			sand			gravel			cobbles	
	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse		

AS-1289

liquid limit %	
plastic limit %	
plasticity index %	
linear shrinkage %	
particle density t/m^3	
natural moisture %	



classification

(SP) SAND- fine to coarse,
 yellow brown, some fine to
 medium gravel, trace of fines

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borehole no	BH3
sheet	1 of 1

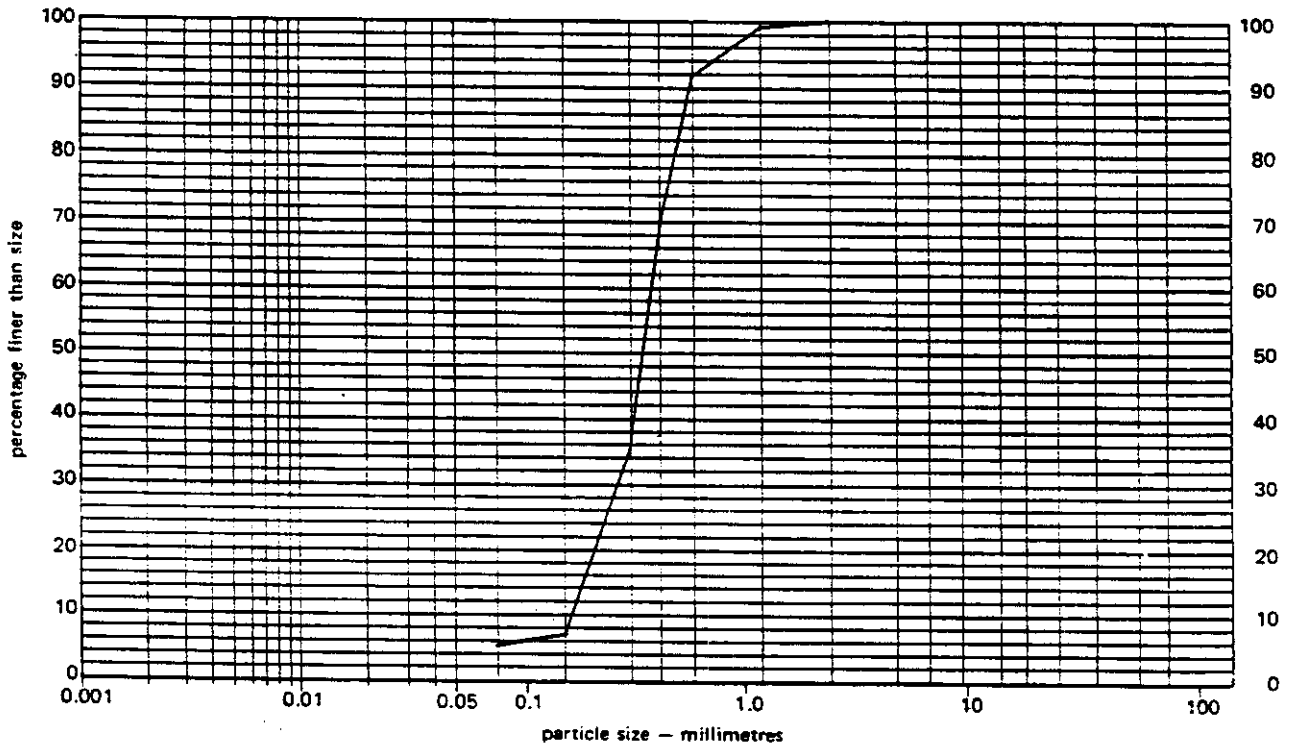
particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21-12-90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC

sample identification	BH3	depth	2.6 - 3.05m
test procedure	AS1289 C6.1 - 1977		

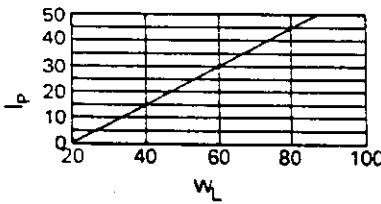
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%	-
plastic limit	%	-
plasticity index	%	-
linear shrinkage	%	-
particle density	t/m ³	-
natural moisture	%	-



classification
 (SM/SP) SAND, fine to coarse,
 yellow-brown, some fines

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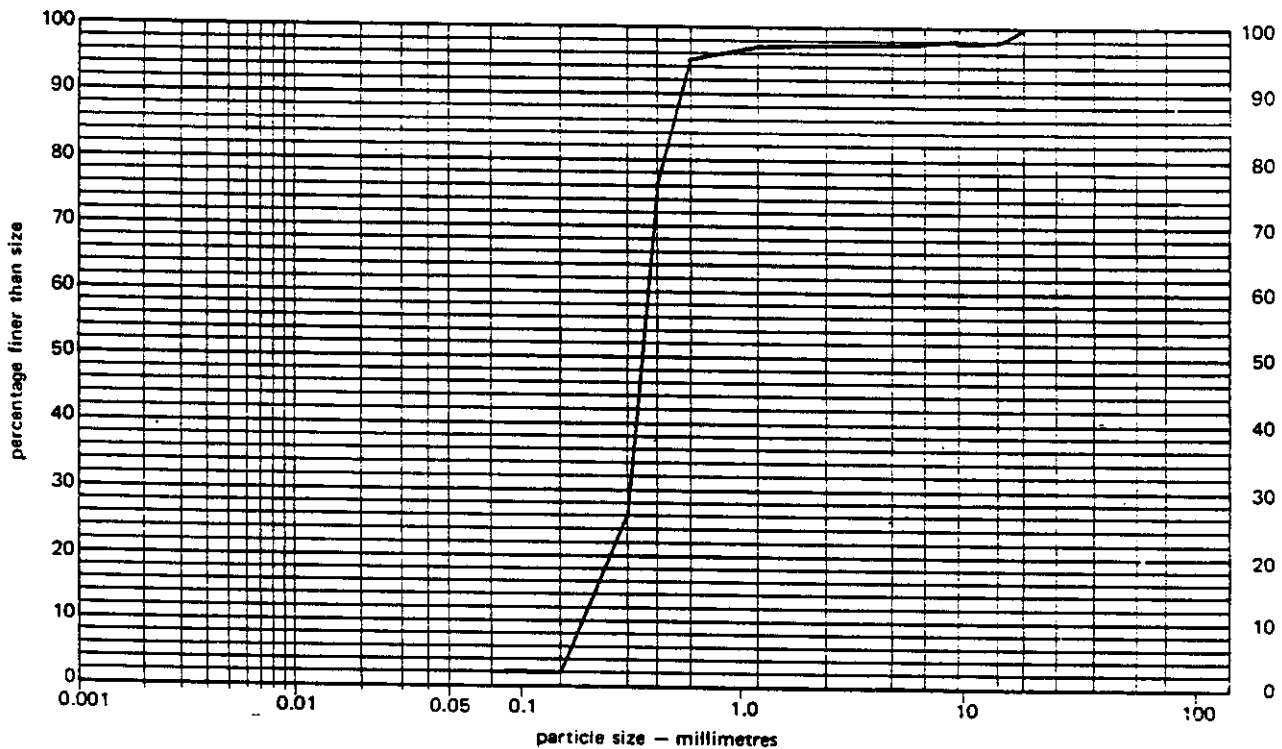
borehole no
 BH4
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEAN TO COLLAROY BEACH	checked by	GC
sample identification	BH4	depth	1.10-1.55m
test procedure	AS1289 C6.1 - 1977		

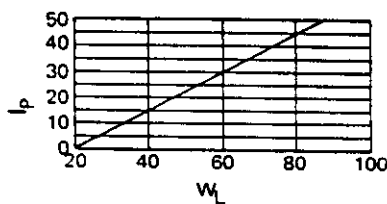
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%	
plastic limit	%	
plasticity index	%	
linear shrinkage	%	
particle density	t/m ³	
natural moisture	%	



classification

(SP) SAND - fine to coarse,
 yellow brown, trace of fines,
 trace of fine gravel.



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borehole no
 BH4
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

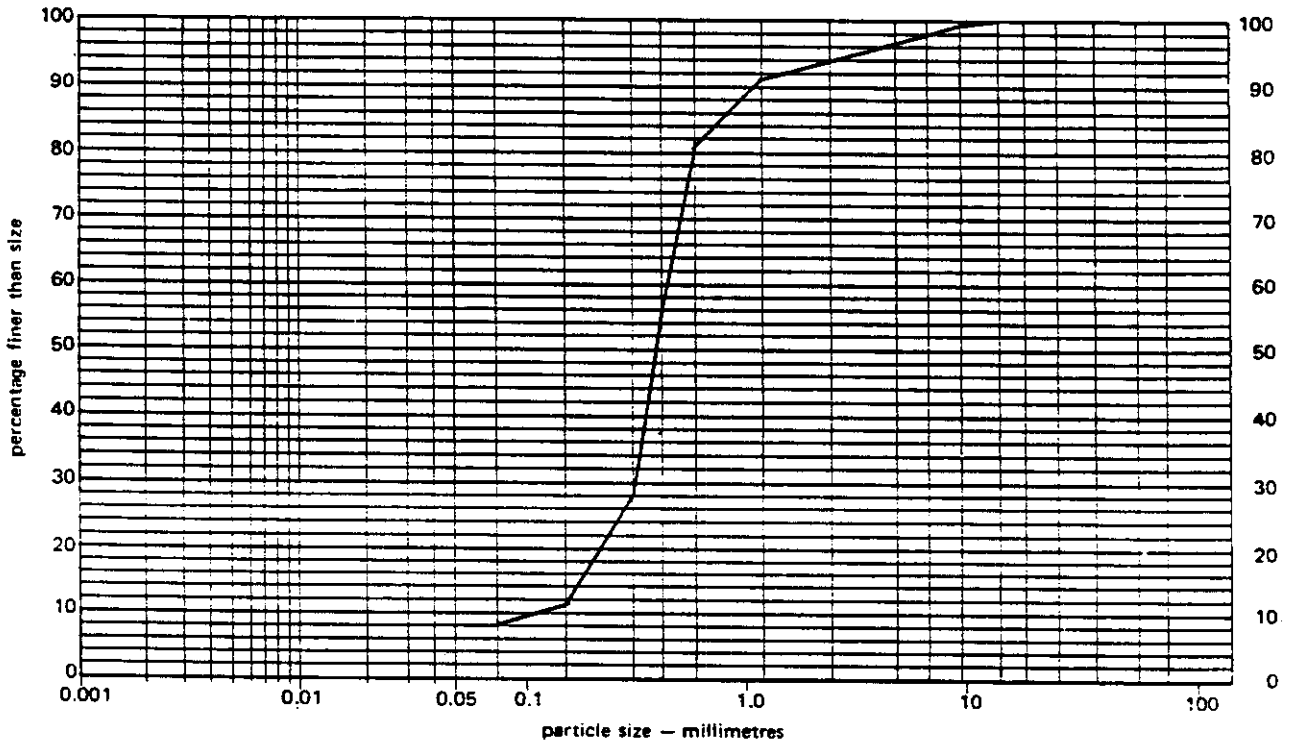
client GEOMARINE PTY LTD
 principal WARRINGAH SHIRE COUNCIL
 project BEACH DEVELOPMENT DESIGN CRITERIA
 location NARRABEEN TO COLLAROY BEACH

job no S9425/1
 date 21/12/90
 tested by GC & KK
 checked by GC

sample identification BH4
 test procedure AS1289 C6.1 - 1977

depth 4.1 - 4.5m

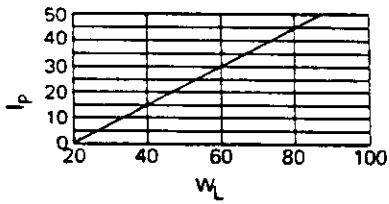
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit %	
plastic limit %	
plasticity index %	
linear shrinkage %	
particle density t/m ³	
natural moisture %	



classification

(SC/SP) SAND - fine to coarse, yellow-brown, some fines, some fine to medium gravel.

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8 1 91



borehole no
 BH5
 sheet 1 of 1

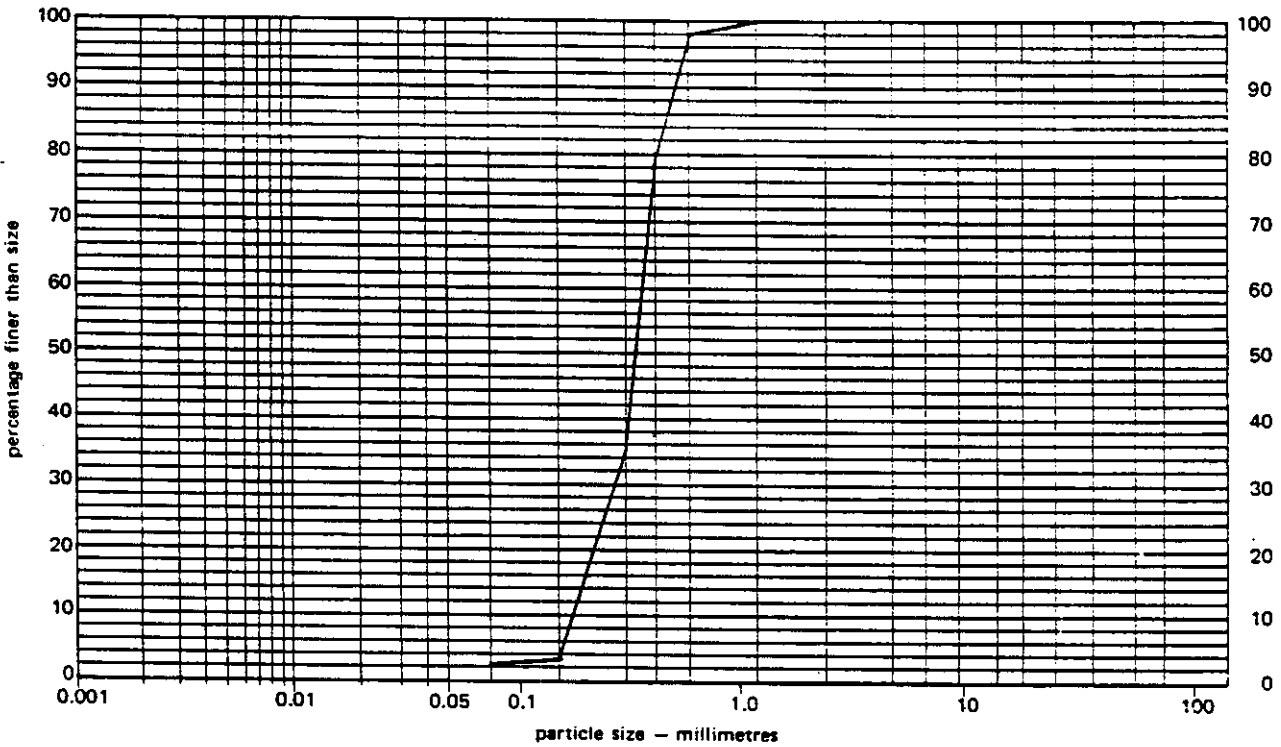
particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC

sample identification	BH5	depth	2.6-3.05m
test procedure	AS1289 C6.1 - 1977		

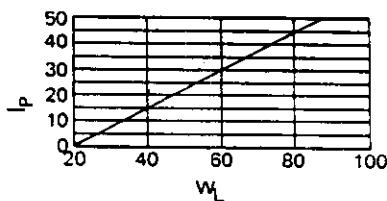
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%	
plastic limit	%	
plasticity index	%	
linear shrinkage	%	
particle density	t/m ³	
natural moisture	%	



classification

(SP) SAND - fine to coarse,
 yellow brown, trace of fines

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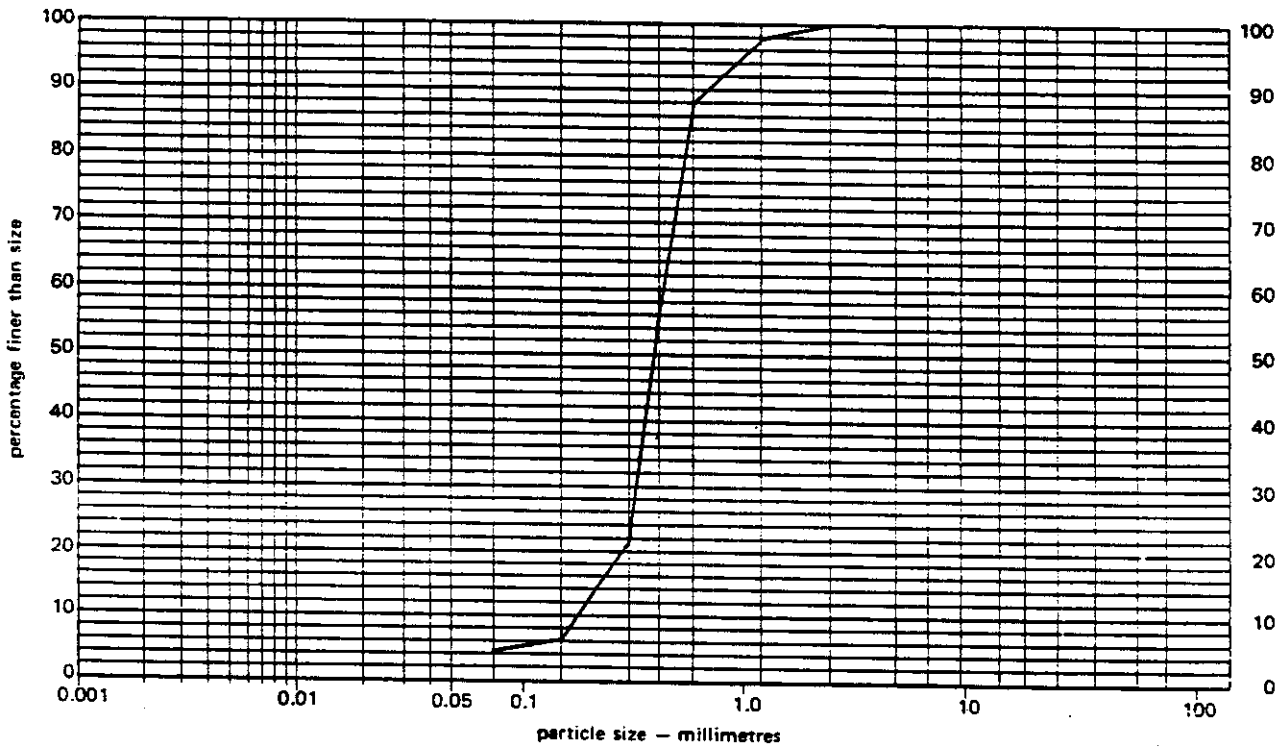
borehole no
 BH5
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC
sample identification	BH5	depth	5.6 - 5.95m
test procedure	AS1289 C6.1 - 1977		

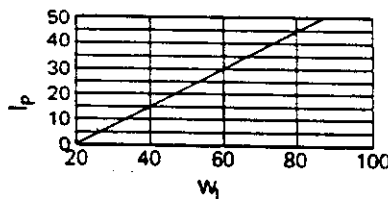
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%	
plastic limit	%	
plasticity index	%	
linear shrinkage	%	
particle density	t/m ³	
natural moisture	%	



classification

(SP) SAND-fine to coarse,
 yellow brown, trace of
 fines

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borehole no
 BH5
 sheet 1 of 1

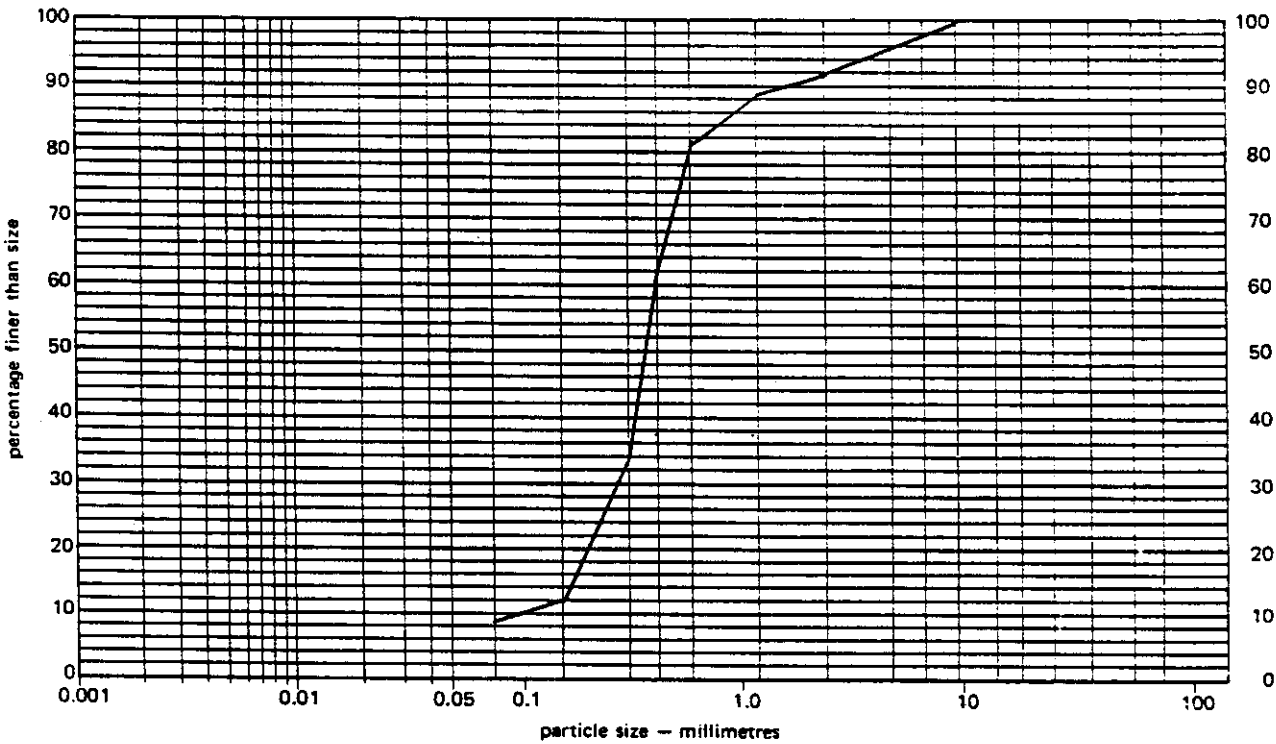
particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC

sample identification	BH5	depth	8.6 - 9.05m
test procedure	AS1289 C6.1 - 1977		

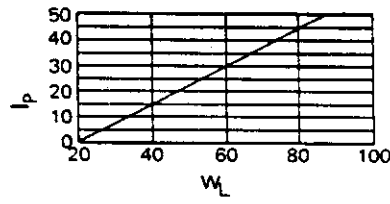
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	0.06	2.0	60			
clay	silt		sand	gravel	cobbles	
	fine	medium	coarse	fine		medium

AS-1289

liquid limit	%	
plastic limit	%	
plasticity index	%	
linear shrinkage	%	
particle density	t/m ³	
natural moisture	%	



classification
 (SC/SP) SAND - Fine to coarse, yellow brown, some fines, some fine to medium gravel. (gravel - cemented sands)

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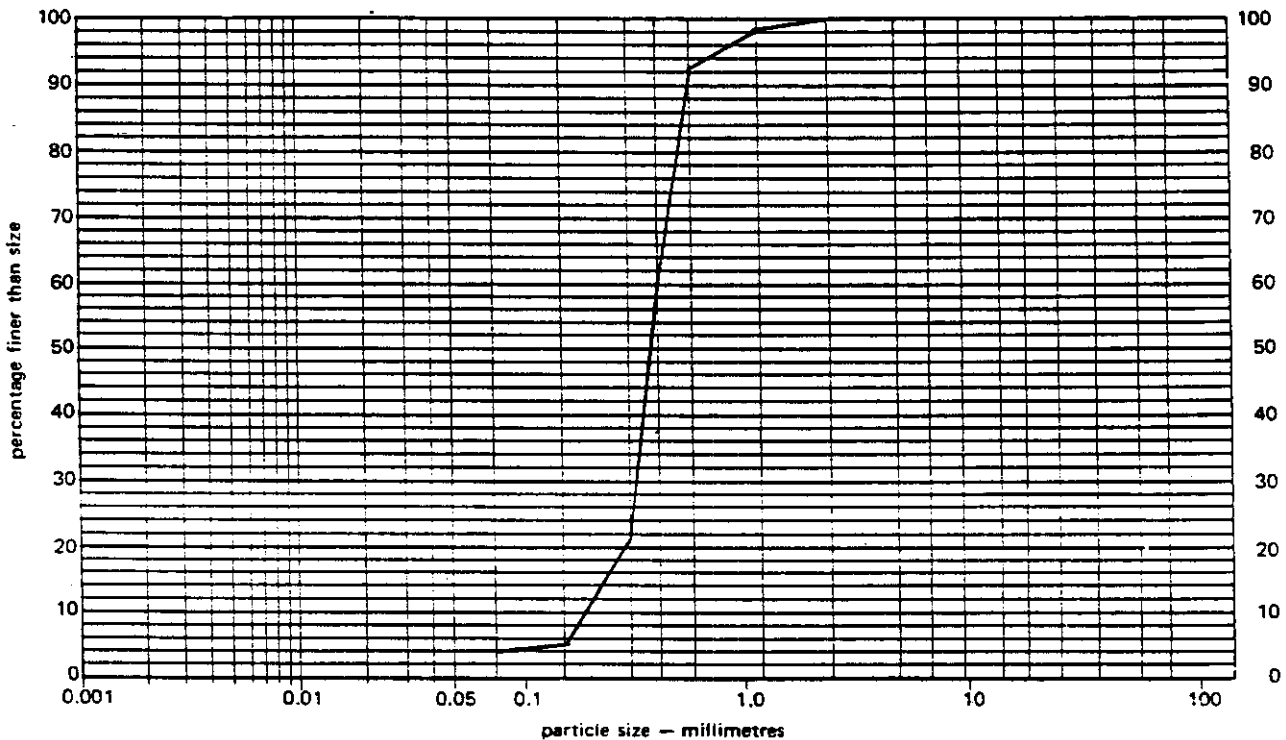
borehole no
 BH6
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC
sample identification	BH6	depth	1.1-1.55m
test procedure	AS1289 C6.1 - 1977		

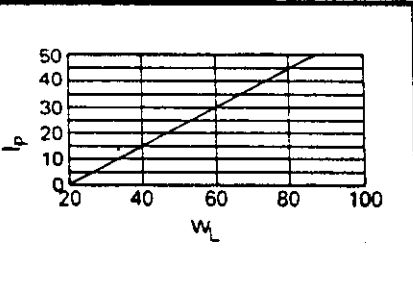
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%
plastic limit	%
plasticity index	%
linear shrinkage	%
particle density	t/m ³
natural moisture	%



classification
 (SP) SAND- fine to coarse, yellow
 brwn, trace of fines

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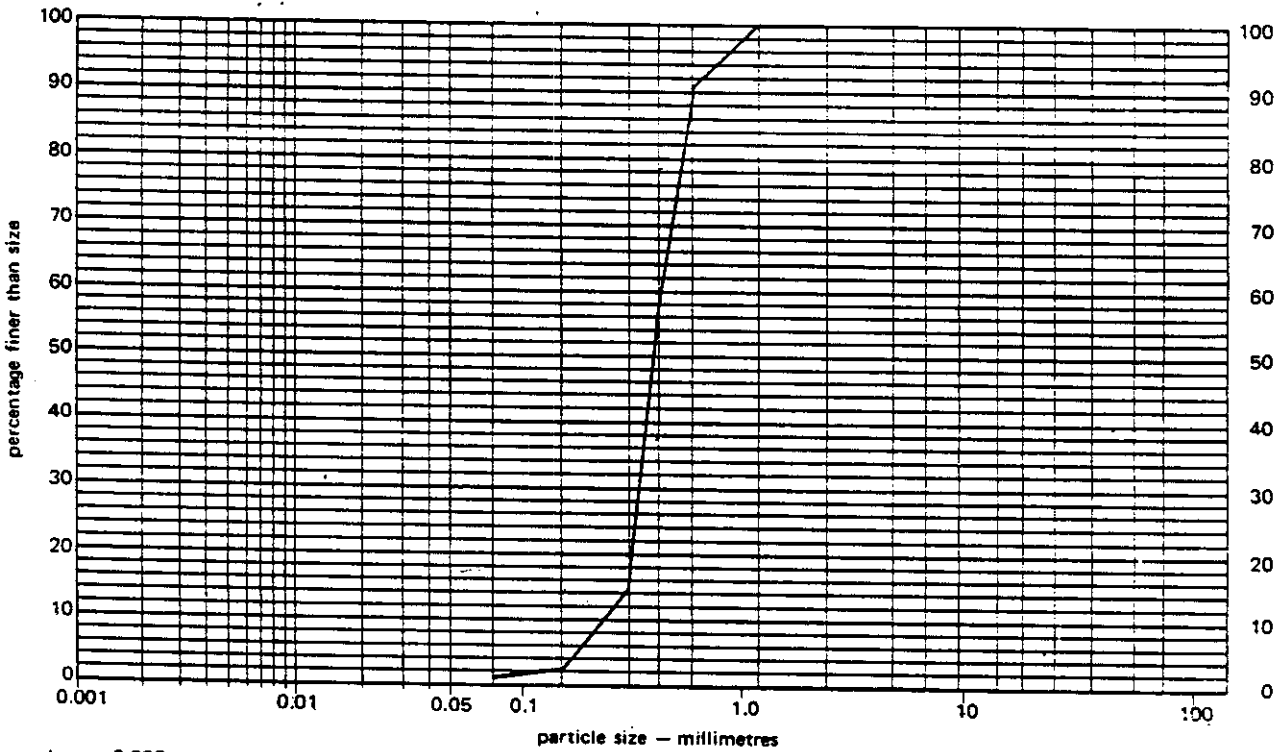
borehole no
 BH6
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC &KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC
sample identification	BH6	depth	4.1-4.55m
test procedure	AS1289 C6.1 - 1977		

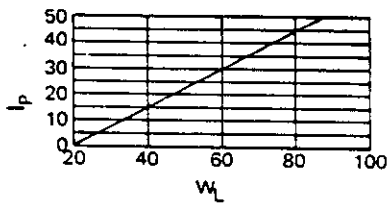
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt		sand			gravel			60	
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%
plastic limit	%
plasticity index	%
linear shrinkage	%
particle density	t/m ³
natural moisture	%



classification
 (SP) SAND - fine to coarse,
 yellow brown, trace of fines

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borehole no
 BH6
 sheet 1 of 1

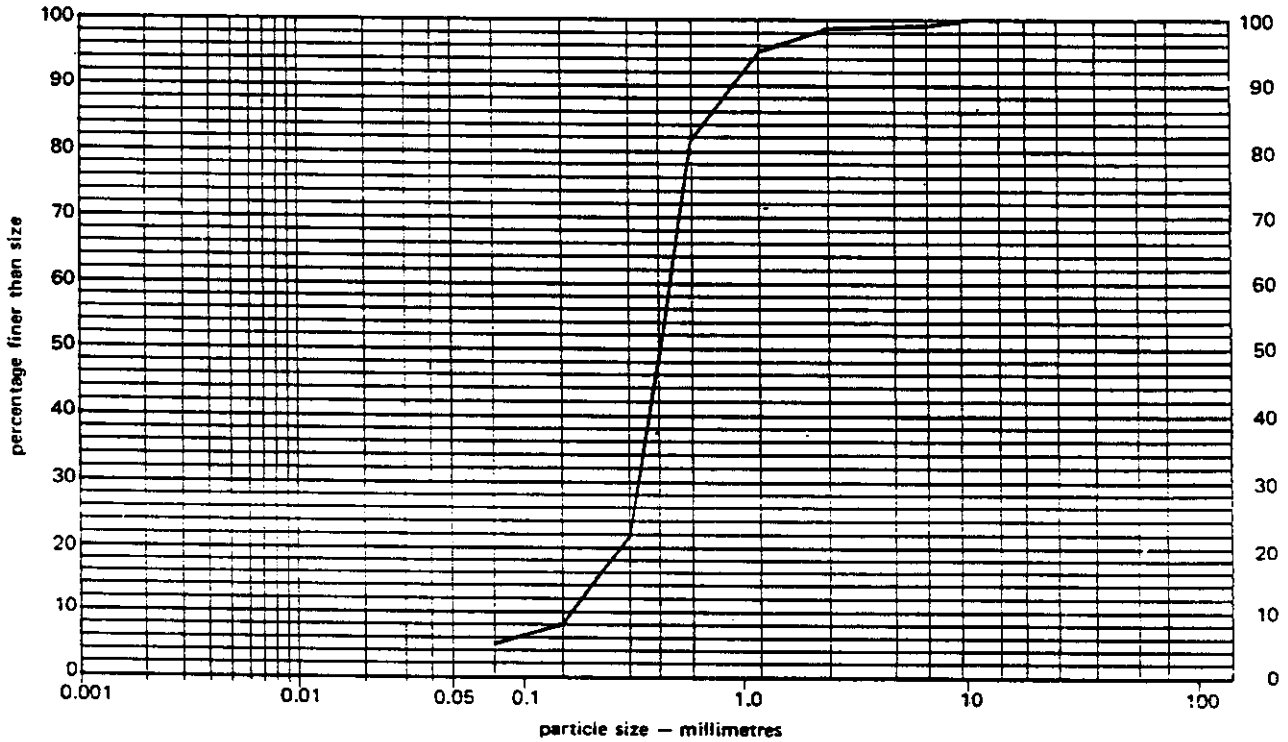
particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC

sample identification	BH6	depth	7.1 - 7.55m
test procedure	AS1289 C6.1 - 1977.		

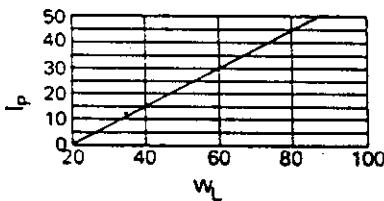
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%
plastic limit	%
plasticity index	%
linear shrinkage	%
particle density	t/m ³
natural moisture	%



classification

(SC/SP) SAND - fine to coarse, yellow brown, trace of fine gravel, some fines.

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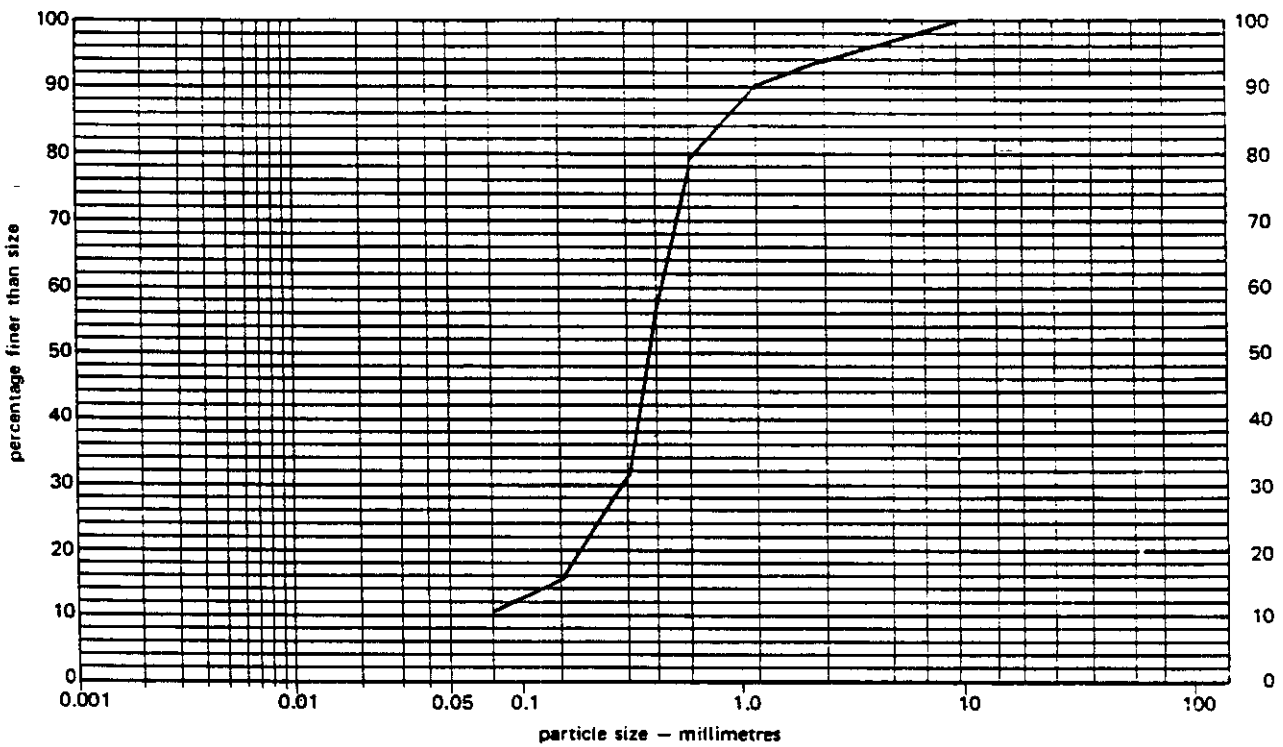
borehole no
 BH6
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC
sample identification	BH6	depth	8.6-8.8m
test procedure	AS1289 C6.1 - 1977		

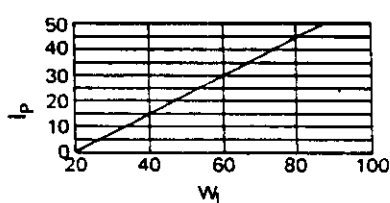
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
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0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%	
plastic limit	%	
plasticity index	%	
linear shrinkage	%	
particle density	t/m ³	
natural moisture	%	



classification
 (SC/SP) SAND - fine to coarse,
 yellow brown, some fine to
 medium gravel, some fines
 (Gravel= cemented sands)

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 Authorized Signature *James Russell*

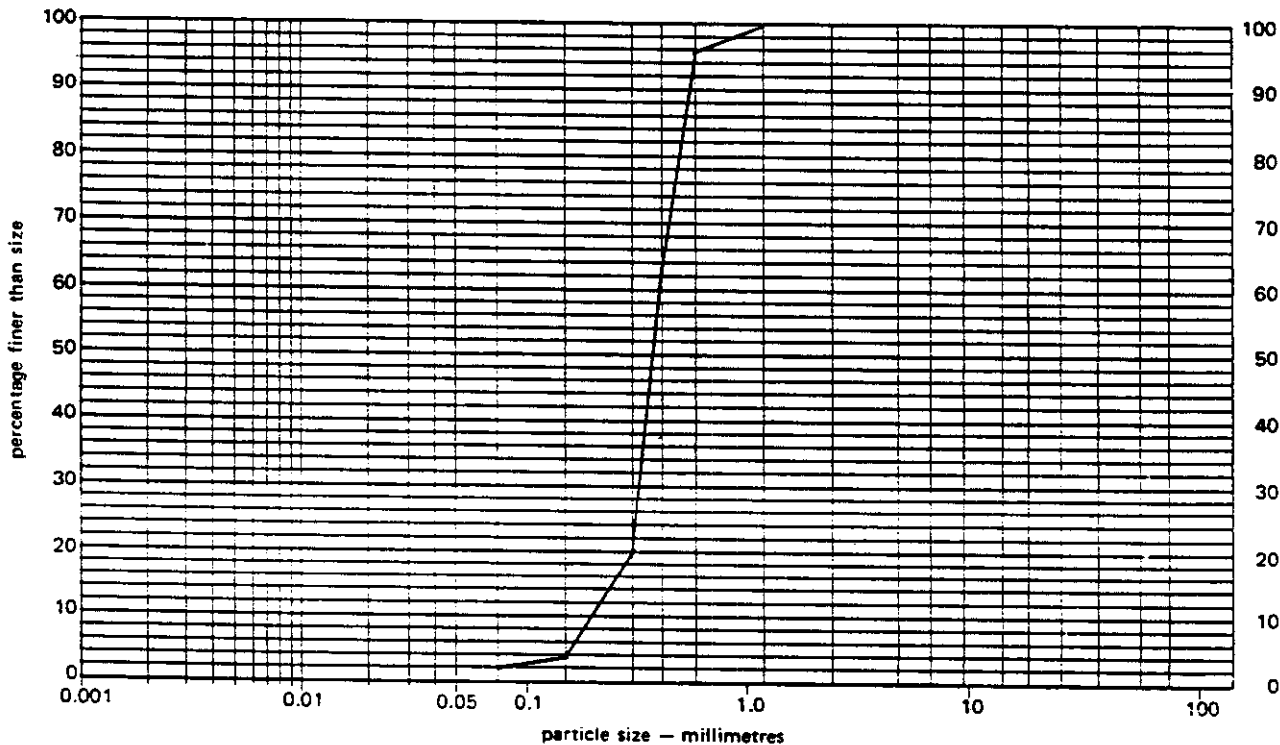


borehole no
 BH7
 sheet 1 of 1

particle size distribution

client	GEOMARINE PTY LTD	laboratory	SYDNEY
principal	WARRINGAH SHIRE COUNCIL	job no	S9425/1
project	BEACH DEVELOPMENT DESIGN CRITERIA	date	21-12-90
location	NARRABEEN TO COLLAROY BEACH	tested by	GC & KK
		checked by	GC
sample identification	BH7	depth	2.6 - 3.05m
test procedure	AS1289 C6.1 - 1977		

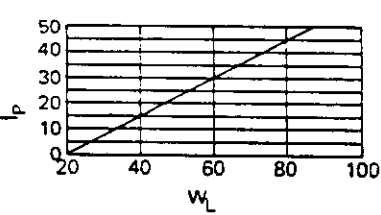
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---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit %	-
plastic limit %	-
plasticity index %	-
linear shrinkage %	-
particle density t/m ³	-
natural moisture %	-



classification
 (SP) SAND, fine to coarse,
 yellow-brown, trace of
 fines.

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8 1 91
 Authorised Signature *James Russell*



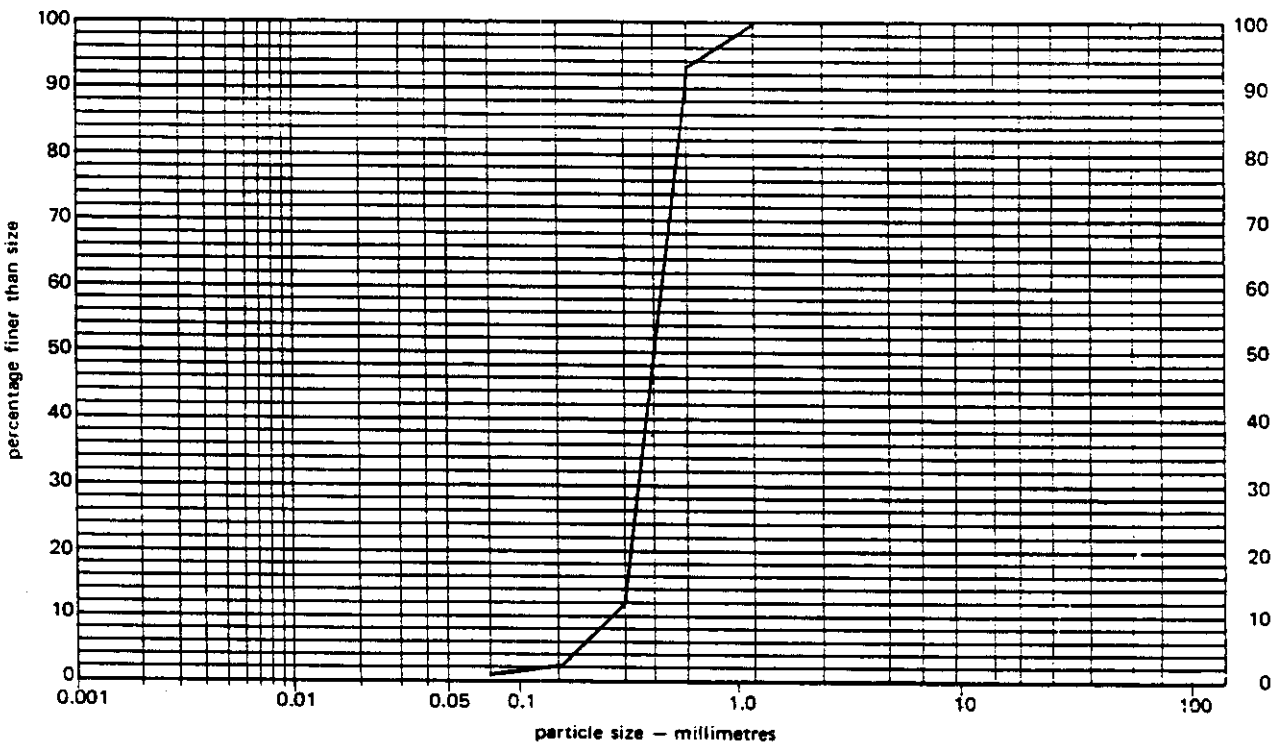
borehole no
 BH7
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC
sample identification	BH7	depth	7.1-7.55m
test procedure	AS1289 C6.1 - 1977		

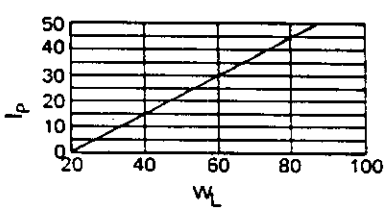
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---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%	
plastic limit	%	
plasticity index	%	
linear shrinkage	%	
particle density γ_m^a		
natural moisture	%	



classification
 (SP) SAND - fine to coarse,
 yellow brown, trace of fines

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 Authored Signature *James Russell*



borehole no
 BH 8
 sheet 1 of 1

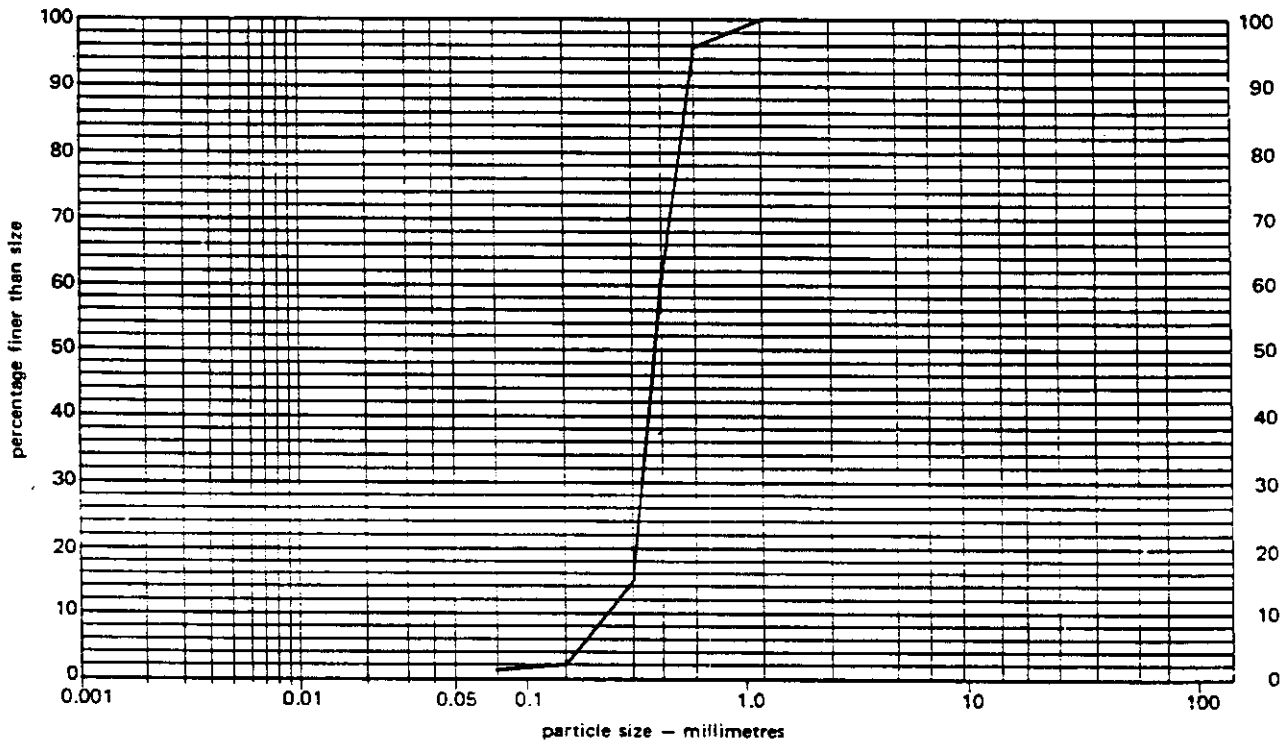
particle size distribution

laboratory SYDNEY

client	GEOMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21-12-90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC

sample identification	BH8	depth	2.6 - 3.05m
test procedure	AS1289 C6.1 - 1977		

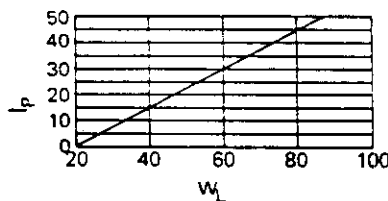
AS sieve size	75µm	150µm	300µm	425µm	600µm	1.18mm	2.36mm	4.75mm	6.7mm	9.5mm	13.2mm	19mm	26.5mm	37.5mm	53mm	75mm	150mm
---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%	-
plastic limit	%	-
plasticity index	%	-
linear shrinkage	%	-
particle density	t/m ³	-
natural moisture	%	-



classification

(SP) SAND, fine to coarse,
 yellow-brown, trace of
 fines.

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Authorised Signature

8 1 91



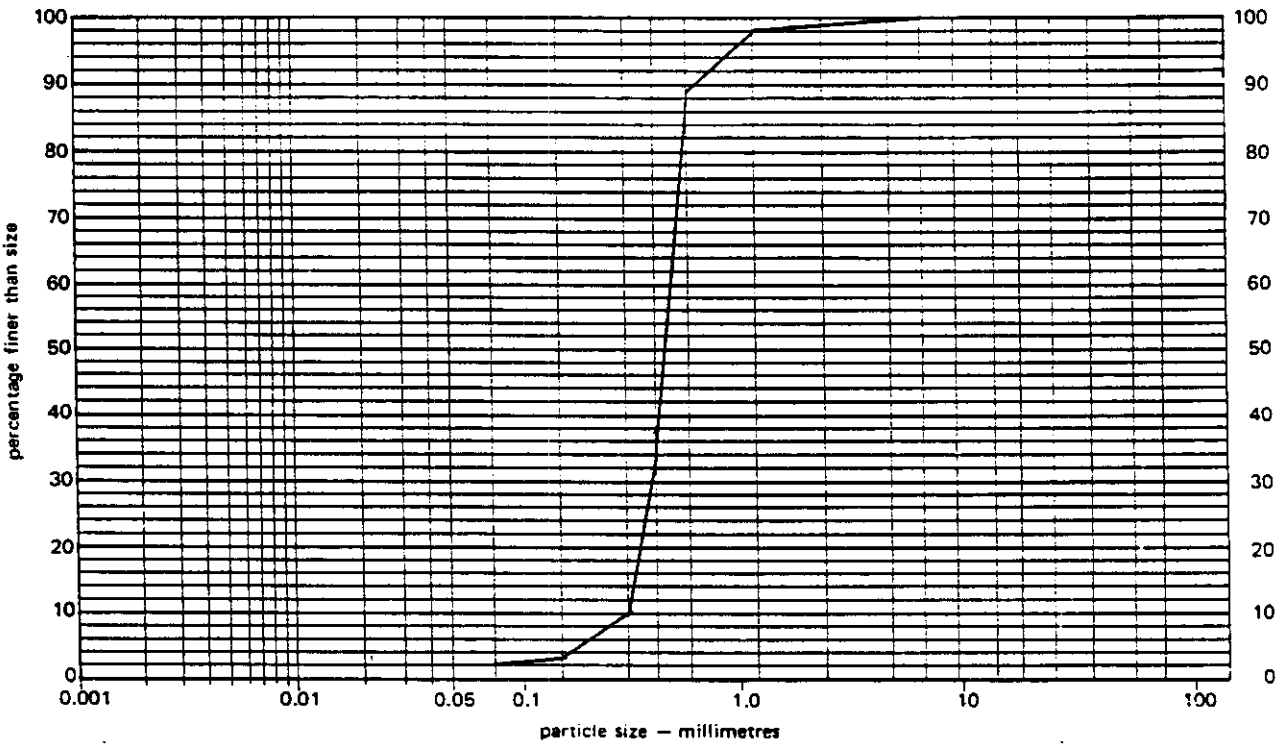
borehole no
 BH8
 sheet 1 of 1

particle size distribution

laboratory SYDNEY

client	GOEMARINE PTY LTD	job no	S9425/1
principal	WARRINGAH SHIRE COUNCIL	date	21/12/90
project	BEACH DEVELOPMENT DESIGN CRITERIA	tested by	GC & KK
location	NARRABEEN TO COLLAROY BEACH	checked by	GC
sample identification	BH8	depth	5.6-6.05m
test procedure	AS1289 C6.1 - 1977		

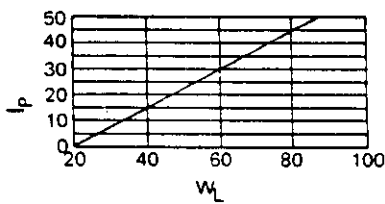
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---------------	------	-------	-------	-------	-------	--------	--------	--------	-------	-------	--------	------	--------	--------	------	------	-------



0.002	silt			sand			gravel			60
clay	fine	medium	coarse	fine	medium	coarse	fine	medium	coarse	cobbles

AS-1289

liquid limit	%	
plastic limit	%	
plasticity index	%	
linear shrinkage	%	
particle density	t/m ³	
natural moisture	%	



classification

(SP) SAND - fine to coarse,
 yellow brown, trace of fine
 gravel, trace of fines

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Authorised Signature

8 1 91
James Russell

Appendix D

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Appendix E

Technical Memorandum

88/02

Technical Memorandum 88/02

Greenhouse Sea Level Rise

The consensus of scientific opinion is that a global warming as a result of the *Greenhouse Effect* is occurring and this will impact on both sea level and the distribution of global weather patterns early next century. Further, it is argued that these changes should as far as possible be taken into account in coastal planning and development.

There are no clear guidelines put forward by The Institution of Engineers Australia or by any of the New South Wales Government instrumentalities providing direction as to the manner and extent to which the *Greenhouse Effect* should be considered. Within this framework **GEOMARINE Pty Ltd** has adopted the following policy:

(1) In respect of coastal processes and coastal flooding, the values published for predicted sea level rise by the U.S. National Research Council* have been adopted. These present three scenarios of a high, medium and low rate of sea level rise as detailed by the curves in Figure One and the calculated values at Table 1.

(2) The appropriate sea level rise scenario to be adopted will be linked to

the type of development according to the relationship presented in Table 2. The sea level rise to be considered will apply to the calculation of structure crest levels, design water levels, water levels for the computation of coastal flooding and in setting levels for floors of habitable structures, drainage and services.

(3) The additional storm erosion, foreshore recession and re-alignment of unconsolidated foreshores resulting from both a *Greenhouse* sea level rise and any associated increase in storminess will be accounted for as far as practicable in advice on coastal developments.

(4) The values adopted for a *Greenhouse* sea level rise will be reviewed as and when further information becomes available.

(5) The design values adopted for storm intensity (wind velocity, rainfall intensity, total rainfall) will be revised in accordance with sound engineering practice. Variations to the current design codes, standards and practices as endorsed by the Institution of Engineers Australia will be adopted.

*This **GEOMARINE** Technical Memorandum was issued first in February, 1988. In August, 1989 the Institution of Engineers Australia issued a Policy on The *Greenhouse Effect* (1/5/15). That policy has not resulted in revision of this Technical Memorandum.

**Responding to changes in sea level. National Research Council (U.S.). National Academy Press. Washington, D.C. 1987.

(Modified from U.S. National Research Council, 1987)

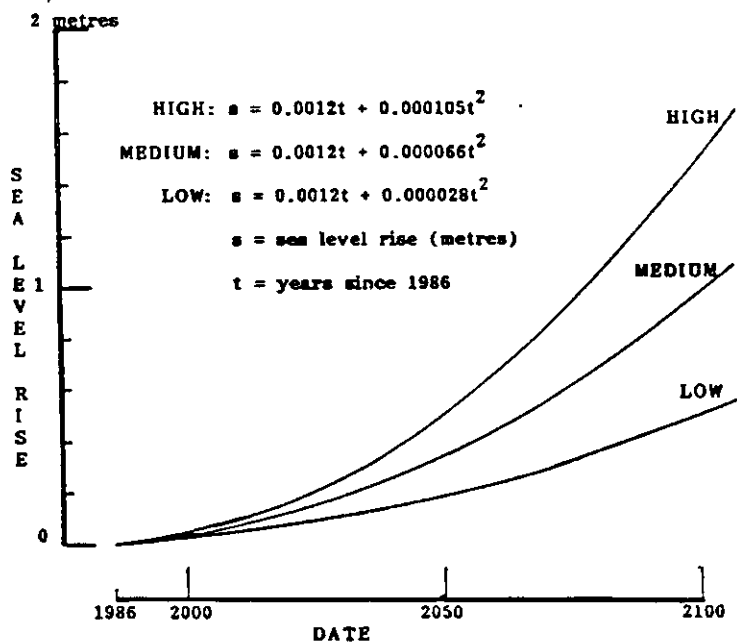


Table 1 - Greenhouse Sea Level Rise Scenarios

SCENARIO	25 YEARS	50 YEARS	100 YEARS
HIGH	0.10m	0.32m	1.17m
MEDIUM	0.07m	0.23m	0.78m
LOW	0.05m	0.13m	0.40m

(Modified from U.S. National Research Council, 1987)

Table 2 - Relating Development and Sea Level Rise

DEVELOPMENT	SEA LEVEL RISE	PLANNING PERIOD
Open space	Low	25 years
Residential	Medium	50 years
Intensive development	High	100years

Greenhouse Sea Level Rise

Technical Memorandum No. 2

Sea Level Rise Scenarios

Figure One